20′ × 120′ STAND-PIPE, ST. AUGUSTINE, FLA.

Frontispiece.

TOWERS AND TANKS

105095

FOR

WATER-WORKS.

THE THEORY AND PRACTICE OF THEIR DESIGN AND CONSTRUCTION.

James Nisbet

BY

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INTRODUCTION.

It it a strange fact to chronicle that, amongst the great mass of scientific literature, there is no distinct treatise upon the design and construction of metallic receptacles or structures, whose province it is to retain a sufficient reserve supply of water, elevated to a proper height and intended to be used in conjunction with other necessary features of a modern water-supply system. Such structures, generally termed "tanks," "water-towers," "stand-pipes," or "towers and tanks," according to their design, are rapidly increasing in number, and are being generally specified in the smaller water-plants, where the economies are to be practised and natural and suitable elevations are unattainable. The popularity of this class of reservoir being on the increase, it would seem that along with the many exhaustive and elaborate discussions of kindred subjects, as hydraulics, hydrostatics, statics, stress, and the metallurgy and physical properties of structural steel, there might be found some work dealing with this now important subject, but so far as the writer is aware, in the entire range of such productions, only the most fragmentary articles are to be found.

The inability to procure definite or reliable information upon the design and construction of such work is probably the cause of the scanty and meagre instructions frequently

appearing in sets of specifications for water-works construction, and the deficiency in this respect has been commented upon by a prominent member of the profession in the following terms:

"The custom has been, to a greater extent than in any other engineering work of like importance, to buy a stand-pipe much as a barrel of flour would be bought; the contract or agreement would be for a stand-pipe so high and so wide, the material and workmanship to be first-class in every respect."

Without previous experience and unable to secure any degree of exact information as to the best practice for stand-pipe design, it would be amusing, if not so serious a matter, to compare the emaciated paragraph, its stock phrases and blanket clauses, so lax that any "rule of thumb" boiler-maker can safely provide almost anything in the shape of a tank, provided it holds together and does not leak too badly, with the plethoric clause, wasting much good paper and printer's ink in padding the specifications to give an important appearance to the technical description dealing with requirements for "cast-iron pipe," which probably gets its first inspection when the pressure is applied from the pumping engines.

Observing this condition of affairs, and having experienced personally the difficulties to be encountered in securing data for work of this sort, the writer offers no further excuse for tendering to others the result of his experience and research, in the hope that this may be of some service to those who, like himself, have had to grope toward the light.

In the treatment of this subject, it is intended to avoid as much as possible elaborate calculations and deductions based upon problematical theories and conditions, and to present such facts as may have been verified, freed, as nearly as may be possible, from the tons of mathematical rubbish which

Trautwine asserts frequently bury the simplest truths. Necessarily, a great portion of such productions as this is compiled from the experience and work of others, and it is the intention of the writer to give to all such due credit, and it is his hope, also, that this, with such record of personal experience as is here offered, may be of service to beginners and of some use to the profession in general.

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TOWERS AND TANKS FOR WATER-WORKS.

CHAPTER I.

BRIEF MENTION OF ANCIENT AND MODERN WORKS.

AMONGST the earliest evidences of a prior civilization, ruined aqueducts, varying in design and extent, indicate the appreciated necessity of public water-supply for populous communities. During the reign of the Jewish King, Solomon, extensive reservoirs or pools were designed and constructed, which to the present time bear his name and testify to the wisdom accredited him, continuing, after the lapse of ages, to deliver a supply of pure water to the citizens of Jerusalem.

The important works constructed under the Cæsars present a good example of the excellence attained by the hydraulician and the general requirements in the matter of water-supply of that day, whilst in the New World, amid the wreck of a more remote antiquity, are to be found examples of the genius of that mysterious race, the Aztec, and its application toward the development of this most important factor in the progress of nations.

Recognizing and putting into practical use the principles of the great natural law of the flow of liquids impelled by gravity, convenient mountain streams and brooks were impounded and led down the hillsides by open channels or aqueducts for the convenience of the people.

In scope such works were necessarily limited by topographical conditions, and permitted only the application of the principles governing what is to-day known as "The Gravity System."

For centuries this method of water-distribution prevailed, varied and modified to suit different conditions, but being shorn from time to time of original crudities, and participating in the general advance toward a higher civilization, the system has reached a high degree of efficiency.

The wonderful advancement of the present epoch in scientific knowledge and mechanical development has made possible the economical production and transmission of power, along with which has come the knowledge of, necessity for, and advantage to be derived from the employment of mechanical means and methods for the accomplishment of required results by other than the primitive principles of gravity flow.

The reference to advantages to be derived from the employment of artificial methods as applied to water-distribution, rather than the utilization of natural agencies, is relative, and is intended to apply only to a broadening of the possibilities; for in the consideration of the question of general or particular source of water-supply, the first investigation should deal with the possibility of procuring a gravity flow, and all subsequent propositions should be referred to the cardinal principle and initial hypothesis that for economy, efficiency, and consequent desirability, Nature's methods take precedence over mechanical means.

Methods of Distribution.—Since the application of scientific methods to natural forces, the problem of water-distribution may be broadly separated into three general schemes or systems—"The Gravity," "The Reservoir," and "The Direct"—each showing particular advantage in individual cases.

Of the first of these, for the purposes of this discussion, possibly enough has been said.

The second, under a multiplicity of design, has for its object the mechanical elevation of water from a lower to a higher level, and its storage in basins or reservoirs of sufficient size and elevation to answer all of the requirements.

The third, or "Direct," scheme distributes the water by a constant, applied mechanical pressure to the contemplated points of delivery. In this monograph, a subdivision of the second of these broad methods will be discussed, as its scope is intended to cover the architectural design; materials and methods of constructing and erecting elevated storage-reservoirs, which of late years have played an important part in the general economy of most water-works designs.

Reservoir System Discussed.—The detail of such construction is subject to local condition, and ranges from designs for small tanks elevated upon supporting columns to immense reservoirs for the water-supply of great cities. In the general scheme of a water-supply system the elevated reservoir serves a dual purpose; providing for a surplus supply to be utilized as required, as well as permitting a temporary suspension of the mechanical operations of the plant; its further purpose is its ability to relieve internal pressures, acting in this capacity as a regulator or relief-valve to the entire system of distribution. Considered simply as a receptacle for elevated storage, its purpose and principles are obvious.

In the natural exercise of the functions of an automatic safety-valve, the results are similar to those produced by an air-chamber, closely connected to the pumping machinery. The force exerted in the intermittent action of an enclosed column of water compressed or impelled by the forward movement of the pistons or plungers of the pumping-engine, acts as a "ram," producing rupture, according to the intensity of the force exerted, to pipe-mains, connections and joints. This stress may be relieved and the shock regulated by providing for a discharge of the water under pressure into an

open reservoir whose upper or highest elevation shall be somewhat in excess of the height to which the water would naturally be forced under the stress conditions, otherwise the reservoir will overflow.

Whilst this destructive tendency has been greatly lessened by the use of improved duplex pumping machinery, there is also to be considered in the economy of operation a certain loss of energy due to the force necessary to put in motion the column of water, temporarily suspended at the expiration of each forward stroke of the machinery by the rigid enclosing sides of the pipe-lines. Connections to an open reservoir provide an opportunity for escape and permits an onward movement of the liquid column, relieving the "back pressure," and, through its own momentum, effecting a saving in energy necessary to impel it forward. The relief to the pipe system is to the same extent enjoyed by the pumping machinery, reducing the strains upon the mechanism and the consequent number and extent of repairs, and, more important still, the liability to accident at some critical moment. Any open reservoir or vertical pipe, of whatever diameter and of sufficient height, will afford the desired relief, but it is the usual practice to couple with this desideratum a capacity sufficient for a reserve supply.

The accomplishment of these requirements is generally secured for the larger cities by reservoirs of earth and masonry construction for reasons of economy and permanency, and designed to suit topographical conditions and local demands.

For the same reasons, in all preliminary investigations for the water-supply of the smaller cities and towns, elevated sites suitable for similar construction should be sought and first given careful consideration.

The subject of the theory, details, and construction of such reservoirs has been discussed by such eminent authorities, and so great a volume of scientific and prolix literature has been

devoted to its consideration, that no attempt will be made here to introduce original conclusions, owing to the unlikelihood of the author being able to add anything worthy of receiving consideration.

Introduction of Metallic Reservoirs in the United States.—The historic record of the introduction of metallic reservoirs, if procurable, would be of much general interest, but unfortunately such information is of the most meagre and unsatisfactory character; of more or less doubtful authenticity.

The oldest complete water system installed in the United States is believed to be that erected at Bethlehem, Pennsylvania, in 1754–61, by Hans Christopher Christiansen, at which point two stand-pipes have at different times been constructed. The first of these, a tank 40 × 24 ft. with a capacity of 225,000 gallons, having served its term of usefulness, was abandoned, and a new steel structure replaces it.

Mr. R. E. Neumeyer, superintendent, writes that for some time he has been engaged in procuring data as to the history of this plant, and this he intends giving publicity later, which, it is to be hoped, he will.

In a recent volume of the *Engineering News* there appears a brief article mentioning a stand-pipe erected in the city of New York, by or through the instrumentality of Aaron Burr, in connection with the launching of the Manhattan Company, a banking house, chartered 1799, and in existence at this time. The tank is described as about 35 ft. in diameter by 15 ft. in height, composed of segmental courses of iron castings, with flanged and bolted joints. Each segment is 2½ ft. wide by 5 ft. high, re-enforced by a web, midway, the flanges at the joints being also re-enforced by web angles. An ornamental effect is obtained by beads forming panels on each half of the outer facings of the segmental castings. Four iron hoops are placed around the tank, and the structure is supported by a masonry tower some 15 or 20 ft. in height.

The supply-pipe is 20 ins. in diameter, and is provided with a gate, enclosed in a rectangular chamber, formed by bolting together two flanged iron castings. The following has been subsequently obtained through correspondence:

"Referring to the tank concerning which you make enquiry, and upon the preservation of which is by some erroneously attributed our existence as a corporation, I beg to say in reply to your request for information, that we are unable to furnish any, as the property upon which the tank is situated is, and has been, leased for many years."

According to a compilation of statistics published by "The Manual of American Water-works," for 1897, there are in the United States 3215 complete municipal water-supply plants. Of these 2223 are designed for gravity supply from earth or masonry reservoirs or impounding basins; small wooden tanks, or intended to be operated entirely by direct pressure.

Their Present Extent and Character.—Nine hundred and ninety-two works are equipped with some form of elevated metallic storage-tanks or reservoirs, approximately 30 per cent. of the entire number of plants, whilst 535, or about 50 per cent. of these last have been erected since 1890, the figures pointing clearly along what lines advanced practice in water-works design is tending.

The accompanying table, compiled from the "Manual" for '97, shows to what extent each State has adopted metallic reservoirs, their average diameter and height, and a record of the material used in the construction as far as given. A column of low, or domestic, pressure, and one showing the fire, or emergency, pressure is also added. The summation and average of the columns of figures given is interesting in its indication of the general practice and requirements deemed necessary, and from which the composite stand-pipe is 20.2 ft. in diameter, with a height of 62.7 ft., capable of containing

TABLE NO. 1.

STAND-PIPE STATISTICS.

Name.	Number	Diameter.	Height.	Steel.	Iron.	Low Pressure.	High Pressure.
Maine	21	28	59	....	8	65	99
New Hampshire	8	27	66	....	2	63	86
Vermont	2	32	33	2	....	80	....
Massachusetts	54	31	69	7	22	63	90
Rhode Island	9	30	71	....	4	75	84
Connecticut	4	39	65	1	3	57	85
New York	74	23	79	19	18	65	101
New Jersey	44	20	95	4	13	51	82
Pennsylvania	50	21	81	3	4	70	100
Delaware	4	12	88	....	2	48	108
Maryland	10	16	90	1	2	60	94
District Columbia	none	....	....	....	....	....	....
Virginia	9	23	67	6	2	91	108
West Virginia	15	35	49	3	2	95	127
North Carolina	10	20	100	5	2	47	109
South Carolina	8	16	96	2	4	43	114
Georgia	15	19	89	7	4	59	82
Florida	7	19	100	3	3	63	117
Alabama	12	20	95	3	3	78	112
Mississippi	6	21	95	3	3	58	106
Louisiana	10	14	120	5	3	45	100
Tennessee	8	20	120	5	1	63	101
Kentucky	13	23	104	3	6	62	100
Ohio	54	21	102	27	11	64	108
Indiana	24	17	100	8	3	60	92
Michigan	23	21	80	6	6	56	101
Illinois	60	14	105	23	3	52	107
Wisconsin	20	20	100	7	4	64	113
Iowa	34	14	89	17	5	53	108
Minnesota	12	18	88	6	1	60	130
Kansas	38	15	108	8	10	60	118
Nebraska	46	13	90	14	9	55	122
South Dakota	4	16	90	....	1	65	130
North Dakota	none	....	....	....	....	....	....
Wyoming	none	....	....	....	....	....	....
Montana	1	25	50	....	....	65	110
Missouri	30	14	97	7	6	60	110
Arkansas	18	18	104	2	3	60	100
Texas	59	17	100	18	12	51	118
Colorado	5	23	73	....	1	62	102
New Mexico	none	....	....	....	....	....	....
Arizona	1	14	100	....	....	45	60
Washington	3	15	75	1	....	38	152
Oregon	none	....	....	....	....	....	....
*California	3	....	....	....	....	26	87
Utah	none	....	....	....	....	....	....
Idaho	1	15	80	....	1	85	85
Oklahoma	2	11	127	2	....	67	112

* Many of wood.

150,686 U. S. gallons of water. The average normal pressure is found to be 62.1 lbs. per sq. inch along the distributing system, and this pressure is increased in times of emergency to 104 lbs.

The pressure 62.1, under daily conditions, is equivalent to 143.5 ft. head, therefore the typical stand-pipe has been erected upon some convenient elevation 80.8 ft. above the general points of distribution. These figures have a peculiar interest in that the pressures determined represent those secured by actual design, independent, as is frequently the case with earth and masonry dams and reservoirs, of natural locations. It should be remarked that the compilation includes, under the head of stand-pipes, only cylindrical metallic structures, unsupported except by foundations, but all such have been incorporated in the summation and average, whether intended for storage, regulation, or both combined.

Eccentricity of Design.—In the compilation of the foregoing table, the author was much interested in the special features of individual stand-pipes and tanks, where considerable eccentricity and lack of uniformity exists, as will be shown by the following two examples:

The tank of greatest capacity to this date in the United States is that erected at Greenwich, Conn., designed by Mr. Wm. S. Bacot, C.E., and erected in 1889 at a cost of $12,000, including painting and foundations. This tank is of wrought iron, of 45,000 lbs. specified tensile strength. It is 80 ft. in diameter by 35 ft. in height, and is capable of containing 1,319,472 U. S. gallons. The thickness of plates composing the tank are as follows: bottom, $\frac{5}{16}$ in.; the 1st ring is $\frac{1}{2}$ in. and the top rings $\frac{1}{4}$ in. iron. The joints are fastened with butt-straps. The structure is erected upon a concrete foundation, presumably without anchorage.

In comparison with this colossus, may be cited a standpipe designed and erected in 1876, at Winona, Minnesota,

by Mr. George C. Morgan, C.E. This stand-pipe is a steel cylinder, 4 ft. in diameter by 210 ft. in height, capacity 20,000 gallons. It is enclosed in an outer ring of stone and brick masonry, with a 28-in. annular space. The lower 50 ft. is composed of ½ in. steel plate; the upper rings not stated. The pipe rests upon 18 ft. depth of solid masonry, and the entire construction is supported by timbers arranged to form a platform 24 ft. square, resting upon a sub-foundation of water-bearing sand and gravel.

Of the stand-pipes recorded, 228 are constructed of steel, and 195 of iron, the remaining number uncertain.

Besides the usual form of stand-pipes and tanks, there are many towers and tanks, combination affairs, designed to meet certain conditions where it may seem preferable to carry the effective head of water by open structural supports, rather than by utilizing the lower plate-rings of the shell to enclose the sustaining water-column. These supporting towers are of manifold design and construction, being built sometimes of wood, but more frequently of stone or brick masonry, latterly largely of metal.

Tendency of Modern Practice.—In this connection, the "Manual" editorially says:

"In the design of elevated tanks, curved bottoms have recently been used in a number of instances, and steel supporting towers or trestles are now commonly employed. The elevated tank is now preferred by many engineers to the stand-pipe, it being recognized that in many instances the effective upper 20 or 30 ft. of water can be supported more cheaply, and perhaps safely, by a trestle than by a body of water enclosed in a cylinder. Where high hills are available for sites, and storage is quite as important as pressure, stand-pipes have advantages of their own."

From compilations by the writer, the number of towers and tanks at this time in the United States, utilized by city

water-plants, is 161, generally constructed since 1890. The modern practice is to build them largely of structural or soft steel, and although the procurable data is not so full or complete as the records of stand-pipes in the United States, the general average diameter, height, and capacity is as follows: Diameter, 21.3; height, 36.9; capacity, 101,100 U. S. gallons, supported upon some form of trestle or tower 63.5 ft. On account of temporary service and liability to accident, wooden trestles are now rarely used; stone and brick masonry, although formerly much employed, has recently, on account of cost, been supplanted by metallic towers, principally of steel.

Possibly one of the best modern examples of the tendency toward the erection of the elevated steel tower and tank is that lately constructed at Jacksonville, Florida, at a cost of $10,000, from designs by Superintendent R. M. Ellis, C.E., 1898. This tank is 30 by 45 ft., with conical bottom and cover, surrounded by an ornamental balcony about its base. The tank is supported by 10 6-in. "Z"-bar columns, 100 ft. in height, stiffened with 8-in. "I"-beam ties, and the usual diagonal tie-rods. The steel in the columns is specified to have a tensile strength of 70,000 to 75,000 lbs.; elastic limit 40,000 lbs., with an elongation of 20 per cent. in 8-in., and a reduction at fracture of 40 per cent.

Steel for the tank, straps, rods, and rivets is to be of 60,000 lbs. as a maximum and 56,000 lbs. as a minimum tensile strength; 25 per cent. elongation in 8-in., and 50 per cent. reduction at point of fracture.

No chemical requirements have been made. The joints are made by butt-strap, and the usual requirements for shop-practice and field-work are insisted upon.

CHAPTER II.

THE CHEMICAL AND PHYSICAL PROPERTIES OF STRUCTURAL METAL.

Wrought Iron.—In attempting to discuss the physical and chemical properties of the structural metals, investigation leads by many stages from geological and metallurgical conditions existing in Nature's great laboratory to those finished products daily used in the mechanical arts. Each step in this process of evolution has been given the devoted attention and wisdom of learned scientists, who have contributed to the world the results of their researches in many erudite and voluminous works. It is not within the scope of this volume to do more than attempt to explain certain pertinent features of this complex subject.

In general metallic reservoirs and their supports are constructed of riveted plates and members of iron or steel. Until the last decade iron was almost universally employed, but improved processes of manufacture, reducing at the same time the cost of the product and eliminating the uncertainty of the result, has produced a radical change in this practice, until steel has attained first place as a suitable metal for structural purposes.

In the production of wrought iron, the chemical process is the conversion of crude or "pig" iron into a refined or "merchantable" product by recarburization in a "puddling furnace." For the manufacture of wrought iron, the lower grades of smelted or "pig" iron are employed. The mechanical process of "puddling" is melting and stirring the pig iron,

until the proper degree of oxidation is secured, and then working of the molten metal into a pasty mass or "puddle-ball," which may then be squeezed or hammered into a suitable shape or "bloom" for rolling into bars, technically known as "muck-" or "puddle-bars." When cold, this intermediate product is sheared and bundled into piles of proper sectional area, to which wrought scrap is most commonly added, after which the pile so formed is brought to a welding heat in a "heating-furnace," to be afterward passed through the finishing rolls, becoming "merchant iron," a finished product.

The strength and quality of the finished product depends, naturally, upon the character of the crude iron or "stock," the skill in puddling, or reducing the non-metallic substances, and particularly upon the method and materials used in forming the "pile" to be made into "blooms." All metallic iron contains more or less impurities, and in general such elements as silicon, manganese, carbon, sulphur, and phosphorus appear; the best wrought iron can only be produced from crude iron containing a limited percentage of sulphur and phosphorus, neither of which can be entirely eliminated in the puddling process, a sufficient percentage being left in the product to give unfavorable results if they were able to exert their full effect in the production of crystallization of the fibres of the metallic iron; but the slag, resulting from the other non-metallic impurities, overcomes this tendency in a degree.

The presence of considerable percentages of sulphur produces in the finished iron a condition termed by smiths as "red short"—an inclination to disintegrate or crumble whenever the iron is heated to a working temperature; the cohesion of its particles being affected adversely, the strength of the metal is correspondingly reduced.

The effect of phosphorus, the most detrimental of all the

alloys, is exactly opposite to that produced by excess quantities of sulphur, in that it makes the finished product "cold short," crystalline in appearance, of uncertain strength, and liable to fracture from sudden shock.

That the character or arrangement of the piles has a direct relation to the strength of the product is explained by Campbell as follows: "If the piles were square and were made up of similar pieces of equal length, each layer being at right angles to the one below, and if the bloom were rolled equally in each direction, it is evident that the plate would be as strong in the line of its length as of its breadth; but as the bars from which the pile is formed have been made by stretching the material in one way, and as all practical work requires a piece of greater length than width, it will be seen that the finished product will show much better results when tested in the direction of its length than its width. The result will also depend upon the skill with which the pile has been constructed; upon the perfection of the welding as influenced by the heating and the rapidity of handling, and upon the freedom of the iron from thick layers of slag."

To secure a pure, refined iron, such as should be specified for structural work, it is necessary, first, to require that the chemical components of the crude iron shall be such as under favorable treatment shall give the desired chemical product; secondly, the production of the muck-bar in suitable condition being largely dependent upon the skill of the workmen, other things being equal, preference should be given the product of old and reputable establishments, and this applies with equal force to the finished product, for it is customary in the manufacture of finished iron to utilize large quantities of miscellaneous "scrap iron," purchased in the open market, and this scrap, without any careful or intelligent assortment, is piled with the sheared muck-bar until the proper size and weight bloom are obtained, when it is heated to a welding

heat and rolled into the required shape. The effect of scrap steel or of impure metal within the mass of this pile is to destroy the homogeneïty and produce segregation.

Whilst it is true that sometimes carelessness is responsible for such process of manufacture, more frequently it is the direct result of determined effort upon the part of the manufacturer to cheapen his product by utilizing cheap, miscellaneous scrap metal. When nicked and broken across, or when ruptured under tension, the appearance of this iron, instead of the long, fibrous arrangement of the molecules, indicative of tough, strong material, is crystalline, and the fracture shows a decided brittleness.

According to Prof. J. B. Johnson, there are three well-recognized causes of this crystalline structure, indicative of inferior material.

"First, the so-called wrought iron may have been rolled from fagotted scrap, some of which was probably high-carbon steel, and this portion would show a crystalline fracture.

"Second, the puddle-ball may have been formed under too great a heat (a common fault), so that a portion of it had been actually melted, thus forming of this portion ingot metal or steel, which part would, when cold, be wholly crystalline.

"Third, the puddling process may have been incomplete, when, with a low fire, some of the unreduced pig iron would be removed from the ball, and this would form a coarsely crystalline portion of the final rolled bar."

Steel manufactured for constructive purposes is at present produced by one of two processes: either the "Bessemer" or converter, or by the "open-hearth" or furnace method. From the character of the lining of the converter or furnace being either acid or basic, a further distinctive technical term of "acid" or "basic Bessemer," or "acid" or "basic open-hearth steel" is commercially used.

Physical Differences between Iron and Steel.—The metamorphose of cast iron into steel is produced, as is the case with the refinement of iron, by oxidation as the principal factor. Made from the same material, and transformed by similar chemical agencies, it is not surprising that there is a great similarity of the two finished products, one termed wrought iron and the other structural steel. The difficulty of defining steel and the narrow line separating it from iron is clearly put in the "Manufacture and Properties of Structural Steel" as follows:

"Prior to the development of the Bessemer and open-hearth processes there was little room for disagreement as to the dividing line between iron and steel. If it would harden in water it was steel; if not, it was wrought iron. When the modern methods were introduced, a new metal came into the world. In its composition and in its physical qualities it was exactly like many steels of commerce, and naturally and rightly it was called steel. By degrees these processes widened their field, and began to make a soft metal which possessed many of the characteristics of ordinary wrought iron, and which was not made by any radical changes in methods, but simply by the use of a rich ferro-manganese. Notwithstanding this fact, some engineers claimed that the new metal was not steel, but iron. The makers replied that it was made by the same process as hard steel, and that it was impossible to draw a line in the series of possible and actual grades of product which they made." Mr. Howe, in his "Metallurgy of Steel," says, "The terms Iron and Steel are employed so ambiguously and inconsistently that it is to-day impossible to arrange all varieties under a simple and consistent classification." Continuing to quote from the "Manufacture and Properties of Structural Steel," "It is true, as argued by Mr. Howe, that many of the common products of metallurgy and art shade imperceptibly into one another; but it is surely extraordinary when the

dividing line can not be drawn even in theory, much less in practice; when, wherever it falls, it must divide, not intermediate, but finished products, used in enormous quantities, and blending into one another by insensible gradations, and when every shade of these variations is the subject of rigorous engineering specifications."

It is customary and necessary, in ordering steel, to give a certain margin in filling specifications, and it will be evident, no matter how close this margin is, that if a line could be drawn, it would not infrequently happen that he who ordered ingot iron would receive steel, and he who ordered steel would receive ingot iron.

Many different tests have been proposed at various times for determining the mechanical properties of steels, but although some of them are of value in special cases, the one method of investigation which has become well nigh universal is to break by a tensile stress and measure the ultimate strength, the elastic limit, the elongation, and the reduction of area. Strictly speaking, none of these properties has any direct connection with hardness, and it is also true that in special instances, as with very high carbons, hardening may reduce the tensile strength by the creation of abnormal internal strains; but in all ordinary steels it is certain that hardening is accompanied by an increase of strength, by an exaltation of the elastic limit, and a degree in ductility.

"The fact that common soft steel is materially strengthened by chilling has been widely recognized for many years, but the extent of the alteration in physical properties in the softest and purest metals is not generally understood."

The table on page 17 shows the results of a series of tests made by Mr. H. H. Campbell.

Again from "Manufacture and Properties of Structural Steel":

"The classification by hardening is a dead issue in our

country. It had quietly passed away unnoticed and unknown before the committee of the Mining Engineers had met, and the best efforts of that brilliant galaxy of talent could only produce a kindly eulogy."

EFFECT OF QUENCHING ON THE PHYSICAL PROPERTIES OF DIFFERENT SOFT STEELS.

NOTE.—Bars were 2 in. × $\frac{3}{8}$ in. flats, rolled from 6 in. × 6 in. ingot, and were chilled at a dull yellow heat.

Number of Test-bar		1	2	3	4	5	6
Composition, per cent.	Carbon.	.09	.12	.11	.12	.09	.10
	Manganese.	.44	.32	.43	.32	.39	.16
	Phosphorus.	.011	.004	.010	.004	.017	.010
	Sulphur.	.033	.027	.010	.027	.031	.019
Ultimate strength, pounds per square inch	Natural.	49390	48960	48960	48260	49760	46250
	Quenched.	66080	65670	66300	63640	62280	58380
Elastic limit, pounds per square inch.	Natural.	33220	33390	33010	32340	31040	29830
	Quenched.	47310			50170	46580	40500
Elastic ratio, per cent.	Natural.	67.26	68.20	67.42	67.01	62.38	64.50
	Quenched.	71.60			78.83	74.79	69.38
Elongation in 8 in., in per cent.	Natural.	29.75	31.00	32.50	32.50	31.25	37.75
	Quenched.	18.75	16.25	15.00	17.75	23.75	27.50
Reduction of area, per cent.	Natural.	50.80	52.50	54.10	55.75	49.00	68.38
	Quenched.	56.50	63.27	63.47	64.47	65.15	68.97

"Strictly speaking, some mention must be made of hardening in a complete and perfect definition, for it is possible to make steel in a puddling-furnace by taking out the viscous mass before it has been completely decarburized; but this crude and unusual method is now a relic of the past, and may be entirely neglected in practical discussion.

"No attempt will be made here to give any iron-clad formula, but the following statements portray the current usage in our country:

"(1) By the term 'wrought iron' is meant the product of the puddling-furnace or the sinking-fire.

"(2) By the term 'steel' is meant the product of the cementation process, or the malleable compounds of iron made in the crucible, the converter, or the open-hearth furnace."

Effect of Heating.—The changes produced in the physical properties of steel through reheating and chilling by quenching are radical; little less so is the effect produced by annealing, or the tempering of steel by reheating as in shop-work, where the metal, after being heated for rolling or bending, is allowed to cool gradually.

The average extent of the changes thus produced is shown from the tests made by Mr. H. H. Campbell upon specimens both of Bessemer and open-hearth steels, and recorded as follows:

"The decrease in ultimate strength by annealing the Bessemer bars averaged 4175 pounds per square inch in the rounds and 5683 pounds in the flats, while the open-hearth was lowered 5134 pounds in the rounds and 7649 in the flats.

"In this important and fundamental quality the two kinds of steel are very similarly affected, but in other particulars there seems to be a radical difference which is difficult to explain. The elongation of the Bessemer steel is increased by annealing in every case except two, the average being 1.33 per cent., while the open-hearth metal shows a loss in three cases, with an average loss for all cases of 0.21 per cent. This is not very conclusive, but there is a more marked difference in the reduction of area, for in the Bessemer steel there is an increase in the annealed bar in every case varying from 7 to 15.18 per cent., while the open-hearth showed an increase in only three cases, the maximum being 2.81 per cent., and a decrease in five cases, the greatest loss being 7.20 per cent."

The results arrived at by Mr. Campbell after exhaustive tests, comparing the effect upon both Bessemer and open-hearth steels, are as follows: "Annealing is useful in removing the strains caused by distortion, for in such cases the gain in safety more than counterbalances the loss of strength, but it may be accepted as a general rule that steel is in its best condition when it leaves the rolling-mill; that the shop treatment

should retain, as far as possible, the natural qualities of the metal; and that the bar should be heated only when it is necessary to make a permanent bend."

Constructive or soft steel is produced, as has been stated, by one of two processes, the Bessemer and the open-hearth, and a technical classification of the product is determined by the character of the lining employed in the furnace, whether acid or basic. An authentic, brief, and comprehensive statement descriptive of the two general methods of manufacturing structural steels is copied in full from the work so frequently herein quoted, and is as follows:

Bessemer Steel.—"The acid-Bessemer process consists in blowing air into liquid pig iron for the purpose of burning most of the silicon, manganese, and carbon of the metal, the operation being conducted in an acid-lined vessel, and in such a manner that the product is entirely fluid. The way in which the air is introduced is a matter of little importance as far as the character of the product is concerned. . . . The lining is made of either stone or brick, or other refractory material, and is about one foot thick. . . . The blast is kept at a pressure of from 25 to 30 pounds per square inch during the first part of the blow, but, in the case of a very hot charge, or if the slag is sloppy, the pressure must sometimes be reduced to 10 pounds after the flame 'breaks through' (*i.e.*, after the carbon begins to burn), 'to prevent the expulsion of the metal from the nose . . . the heats, whether light or heavy, are usually blown in from 7 to 12 minutes.'"

After the chemical change has taken place whereby the cast iron has become molten steel, the fluid metal is tapped or drawn off into cast-iron moulds, where the metal solidifies so that it may be handled, when it is then called an ingot, and, as such, reheated in a furnace, passed through trains of rolls, as is the case with wrought iron, and rolled into the desired shape.

The basic Bessemer process is identical with that just described, except the converter or furnace is lined with a material that resists the action of the basic slags. Again quoting from the "Manufacture and Properties of Structural Steel": "This lining is usually made of dolomite, but sometimes a limestone is used containing a very small proportion of magnesia. The stone must be burned thoroughly to expel the last trace of volatile matter, and then ground and mixed with anhydrous tar. The highest function of the lining is to remain unaffected, and allow the basic additions to do their work alone, so that the rapid destruction of a basic, as compared with an acid lining, is not due to any necessary part it plays in the operation, but to the fact that there is no basic material in nature which is plastic, and which by moderate heating will give the firm bond that makes clay so valuable in acid practice."

Acid and basic Bessemer steel is sometimes known as converter steel, and depending largely upon the product of the blast-furnace, as well as the possibility of large output, the cost of production of Bessemer steels is considerably less than the product of the open-hearth process, which finds it advantageous to use a considerable proportion of scrap steel, and is more limited in the matter of its output. It is claimed by many authorities that the metallurgical conditions are such that a greater degree of certainty in the production of open-hearth is possible, and, whether this be true or not, the fact remains that the general tendency among engineers and as evidenced by numerous recent specifications, is to give a preference to the open-hearth product over Bessemer steels.

A description of the process of manufacture of the open-hearth product is as follows, and is also from Mr. Campbell's admirable work:

Open-hearth Steel.—"The open-hearth process consists of melting pig iron, mixed with more or less wrought iron, steel, or similar iron products, by exposure to the direct action of

the flame in a regenerative furnace, and converting the resultant bath into steel, the operation being so conducted that the final product is entirely fluid."

As stated, this regenerative furnace steel is classified as acid or basic, depending upon the formation or texture of the lining.

"In one the hearth is lined with sand, and the slag is silicious; in the other the hearth is made of such material that a basic slag can be carried during the operation."

As is the case with wrought iron, the metalloids as carbon, silicon, sulphur, manganese and phosporus affect the finished product, carbon being the least uncertain and detrimental of the alloys, for structural steel being a carbon steel, its presence should possibly not be limited. Also as with iron, the most important of the metalloids are sulphur and phosphorus, the last being the most to be feared. Regarding the effect of sulphur on steel products, Mr. Campbell says: "Nothing is better established than the fact that sulphur injures the rolling qualities of steel, causing it to crack and tear, and lessening its capacity to weld. . . . In the making of common steel for simple shapes, a content of .10 per cent. is possible, and may even be exceeded if great care be taken in the heating, but for rails and other shapes having thin flanges it is advantageous to have less than .08 per cent., while every decrease below this point is seen in a reduced number of defective bars."

Effects of Phosphorus.—The effects of phosphorus, the most potent of all the metalloids for evil, is thus given by Mr. Campbell: "Of all the elements commonly found in steel, phosphorus stands pre-eminent as the most undesirable. It is objectionable in the rolling-mill, for it tends to produce coarse crystallization, and hence lowers the temperature to which it is safe to heat the steel, and, for this reason, phosphoritic metal should be finished at a lower temperature than pure steel in order to prevent the formation of a crystalline

structure during cooling. Aside from these considerations its influence is not felt in a marked degree in the rolling-mill, for it has no disastrous effect upon the toughness of red-hot metal when the content does not exceed .15 per cent."

A discussion of the effects of phosphorus in steel by Howe's "Metallurgy of Steel," and summarized by Mr. Campbell, is as follows:

"(1) The effect of phosphorus on the elastic ratio, as on elongation and contraction, is very capricious.

"(2) Phosphoric steels are liable to break under very slight tensile stress if suddenly or vibratorily applied.

"(3) Phosphorus diminishes the ductility of steel under a gradually applied load as measured by its elongation, contraction, and elastic ratio when ruptured in an ordinary testing-machine, but it diminishes its toughness under shock to a still greater degree, and this it is that unfits phosphoric steels for most purposes.

"(4) The effect of phosphorus on static ductility appears to be very capricious, for we find many cases of highly phosphoric steel which show excellent elongation, contraction, and even fair elastic ratio, while side by side with them are others produced under apparently identical conditions but statically brittle.

"(5) If any relation between composition and physical properties is established by experience, it is that of phosphorus in making steel brittle under shock; and it appears reasonably certain, though exact data sufficing to demonstrate it are not at hand, that phosphoric steels are liable to be very brittle under shock, even though they may be tolerably ductile statically. The effects of phosphorus on shock-resisting power, though probably more constant than its effects on static ductility, are still decidedly capricious. . . ."

Mr. Campbell's conclusion in regard to the effects of phosphorus in the composition of steel, and the limit to be

placed upon its presence, is as follows: "No line can be drawn that shall be called the limit of safety, since no practical test has ever been devised which completely represents the effect of incessant tremor. For common structural materials the critical content has been placed at .10 per cent. by general consent, but this is altogether too high for railroad-bridge work. All that can be said is that safety increases as phosphorus decreases, and the engineer may calculate just how much he is willing to pay for greater protection from accident."

To what extent specifications calling for reduction of this element affect the market price of materials is shown from the following, taken from Prof. Pence's "Stand-pipe Accidents and Failures":

"A recent proposal for the construction of an important stand-pipe in a Western city included bids according to five limitations for phosphorus, running from 0.08 to 0.04 per cent. inclusive. The relative bids on the superstructure for the several grades of steel, taking that for the highest phosphorus limit as unity, were as follows:

Phosphorus Limit.	Relative Bid.
0.08	1.00
0.07	1.03
0.06	1.08
0.05	1.17
0.04	1.23

"The plates were to be 'soft, acid, open-hearth steel,' of 54,000 to 62,000 lbs. per sq. in. in tensile strength; elastic limit, 31,000 lbs. per sq. in.; minimum elongation in 8 inches, 26%; minimum reduction of area, 50%; cold bent flat; and not more than 0.08% phosphorus, and less per cent. as per detailed bid."

Standard specifications for structural steel have been adopted in the United States as follows:

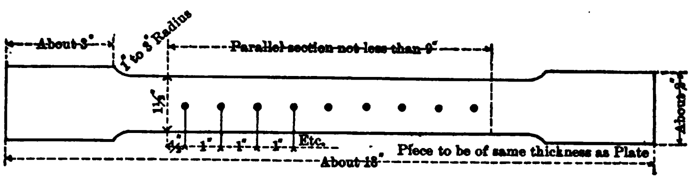
MANUFACTURERS' STANDARD SPECIFICATIONS.

STRUCTURAL STEEL.

1. *Process of Manufacture.*—Steel may be made by either the open-hearth or Bessemer process.

2. *Testing.*—All tests and inspections shall be made at place of manufacture prior to shipments.

3. *Test-pieces.*—The tensile strength, limit of elasticity, and ductility, shall be determined from a standard test-piece cut from the finished material. The standard shape of the test-piece for sheared plates shall be as shown by the following sketch:



On tests cut from other material the test-piece may be either the same as for plates, or it may be planed or turned parallel throughout its entire length.

The elongation shall be measured on an original length of 8 ins., except when the thickness of the finished material is $\frac{5}{16}$ in. or less, in which case the elongation shall be measured in a length equal to sixteen times the thickness; and, except in rounds of $\frac{5}{8}$ in. or less in diameter, in which case the elongation shall be measured in a length equal to eight times the diameter of section tested. Two test-piece shall be taken from each melt or blow of finished material, one for tension and one for bending.

4. *Annealed Test-pieces.*—Material which is to be used without annealing or further treatment is to be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen representing such material is to be similarly treated before testing.

5. *Marking.*—Every finished piece of steel shall be stamped with the blow- or melt-number, and steel for pins shall have the blow- or melt-number stamped upon the ends. Rivet and lacing steel, and small pieces for pin-plates and stiffeners, may be shipped in bundles securely wired together, with the blow- or melt-number on a metal tag attached.

6. *Finish.*—Finished bars must be free from injurious seams, flaws, or cracks, and have a workmanlike finish.

7. *Chemical Properties.*—Steel for railway bridges: Maximum phosphorus, .08 per cent. Steel for buildings, train-sheds, highway bridges, and similar structures: Maximum phosphorus, .10 per cent.

8. *Physical Properties.*—Steel shall be of three grades, *rivet*, *soft*, and *medium*.

9. *Rivet Steel.*—Ultimate strength, 48,000 to 58,000 pounds per square inch.

Elastic limit, not less than one-half the ultimate strength.

Elongation, 26 per cent.

Bending test, 180 degrees flat on itself, without fracture on outside of bent portion.

10. *Soft Steel.*—Ultimate strength, 52,000 to 62,000 pounds per square inch.

Elastic limit, not less than one-half the ultimate strength.

Elongation, 25 per cent.

Bending test, 180 degrees flat on itself, without fracture on outside of bent portion.

11. *Medium Steel.*—Ultimate strength, 60,000 to 70,000 pounds per square inch.

Elastic limit, not less than one-half the ultimate strength.

Elongation, 22 per cent.

Bending test, 180 degrees to a diameter equal to thickness of piece tested, without fracture on outside of bent portion.

12. *Pin Steel.*—Pins made from either of the above-mentioned grades of steel shall, on specimen test-pieces cut at a depth of one inch from surface of finished material, fill the physical requirements of the grade of steel from which they are rolled, for ultimate strength, elastic limit, and bending, but the required elongation shall be decreased 5 per cent.

13. *Eye-bar Steel.*—Eye-bar material, $1\frac{1}{2}$ inches and less in thickness, made of either of the above-mentioned grades of steel, shall, on test-pieces cut from finished material, fill the requirements of the grades of steel from which it is rolled. For thickness greater than $1\frac{1}{2}$ inches, there will be allowed a reduction in the percentage of elongation of 1 per cent. for each $\frac{1}{8}$ of an inch increase of thickness, to a minimum of 20 per cent. for medium steel and 22 per cent. for soft steel.

14. *Full-size Test of Steel Eye-bars.*—Full-size test of steel eye-bars shall be required to show not less than 10 per cent. elongation in the body of the bar, and tensile strength not more than 5000 pounds below the minimum tensile strength required in specimen tests of the grade of steel from which they are rolled. The bars will be required to break in the body, but should a bar break in the head, but develop 10 per cent. elongation and the ultimate strength specified, it shall not be cause for

rejection, provided not more than one-third of the total number of bars tested break in the head; otherwise the entire lot will be rejected.

15. *Variation in Weight.*—The variation in cross-section or weight of more than 2½ per cent. from that specified will be sufficient cause for rejection, except in the case of sheared plates, which will be covered by the following permissible variations:

(*a*) Plates 12½ pounds or heavier, when ordered to weight, shall not average more variation than 2½ per cent. either above or below the theoretical weight.

(*b*) Plates from 10 to 12½ pounds, when ordered to weight, shall not average a greater variation than the following:

Up to 75 inches wide, 2½ per cent., either above or below the theoretical weight.

Seventy-five inches and over, 5 per cent., either above or below the theoretical weight.

(*c*) For all plates ordered to gauge there will be permitted an average excess of weight over than corresponding to the dimensions in the order equal in amount to that specified in the following table.

TABLE OF ALLOWANCES FOR OVERWEIGHT FOR RECTANGULAR PLATES WHEN ORDERED TO GAUGE.

Thickness of Plate.	Width of Plate.			Thickness of Plate.	Width of Plate.	
	Up to 75 in.	75 in. to 100 in.	Over 100 in.		Up to 50 in.	50 in. and above.
1/4 inch.	10 per cent.	14 per cent.	18 per cent.	1/8 up to 5/32	10 per cent.	15 per cent.
5/16 "	8 " "	12 " "	16 " "	5/32 " " 3/16	8½ " "	12½ " "
3/8 "	7 " "	10 " "	13 " "	3/16 " " 1/4	7 " "	10 " "
7/16 "	6 " "	8 " "	10 " "			
1/2 "	5 " "	7 " "	9 " "			
9/16 "	4½ " "	6½ " "	8½ " "			
5/8 "	4 " "	6 " "	8 " "			
Over 5/8 "	3½ " "	5 " "	6½ " "			

Work of International Association.—An effort is being made at this time by the International Association for Testing Materials, to establish international standard specifications for the inspection of iron and steel. Each national branch will contribute to the grand council a committee report, dealing in part with "Determination of Methods of Testing the Homogeneity of Iron and Steel, looking to their Eventual Use for Inspection," and from these reports a new

set of standard specifications may be evolved, but whether in general practice they are to supersede those employed at this time is, of course, entirely conjectural.

During the early portion of the present year the American Division of the International Committee submitted a tentative report, subject to further consideration and discussion before final action is taken at a meeting called for October. So universally has this report been endorsed and so favorably received, that the possibility seems that it will not be materially modified, and that it will receive the approval of the International Committee and spring into general use throughout the civilized world. It is interesting to note that, in treating of structural material, its introductory, defining the process of manufacture, advocates a radical departure from the "Manufacturers' Standard Specifications" in that it eliminates the Bessemer process of manufacture, requiring that "Steel shall be made by the open-hearth process." This is not such a radical departure as it would seem upon the surface, as prior to this report, the tendency toward a preference for this product was everywhere in evidence, and had become a commercial possibility through the erection of numerous open-hearth plants of large capacities, an immense impetus having been given this method of production by the successful commercial development of the open-hearth continuous process, permitting the use of fluid metal from blast-furnaces, mixers, and cupolas. Altogether, the "signs of the times" distinctly point to the increased production of open-hearth steel for structural materials, possibly to the complete elimination of the Bessemer method of manufacture.

CHAPTER III.

THE USE OF IRON.

NOTWITHSTANDING the inability of metallurgists to determine with certainty the precise point in its evolution when iron is converted into steel, and conceding scientific uncertainty as to technical definition, the well-known characteristics of iron and steel exhibit radical differences, and practical metal-workers seldom err in determining each with certainty; therefore comparison is entirely pertinent in considering both metals as materials for stand-pipe construction, and the individual merits of each, referring to general utility, fitness, and comparative cost, should receive consideration.

Until 1880 iron plate was used almost exclusively in the construction of metallic reservoirs, although a steel pipe is recorded as having been erected as early as 1876, about which time the commencement of the steel industry in the United States may be said to have dated. From that time the introduction of metallic members in structures slowly and timidly advanced, criticised at each step; but, profiting by each failure, overcame the difficulty until at the present time few mills continue the practice of rolling iron shapes and plates for structural work, and specifications calling for ferric members are now practically obsolete.

The United States Statistical Bureau of the Treasury Department, for the year 1899, places the United States at the head of the steel and iron producing countries of the world, with a record of 13,620,703 tons of pig iron produced, of which 78.1 per cent., or 10,639,857 tons, was converted into steel.

The Change to Steel.—The underlying cause for a change so radical as to amount to an industrial revolution, is the appreciation and realization of the commercial and constructive value of steel, leading to scientific advance constantly improving the physical and chemical properties, whilst the increased demand introduced new facilities for reducing the price of the product below that of commercial wrought iron. As has been stated, at this time, of the 992 metallic reservoirs in the United States, 220 are of iron and 292 of steel, leaving 480 undefined. Whilst these records give only a small excess of steel as compared with iron structures, the increased use of steel is more apparent when it is considered that only within the past few years has steel been recognized as a suitable metal for such work.

Record of Failures.—From the best procurable records amongst the entire number of metallic reservoirs of water-supply plants in this country, there are recorded 45 partial and complete failures and collapses, 13 of which are credited to steel structures, whilst only 5 known to have been built of iron plates have failed. From this it would seem that steel tanks are more liable to collapse than iron ones, but this fact should only be admitted conditionally and after consideration of the causes inducing the failures.

Of the 13 complete and partial failures attributed to the list of steel tanks, the date of erection and failure shows the majority of them to have been constructed during what might be termed the experimental stage of steel-production, as, for instance, chemical analysis of the steel used in four of these tanks shows a large proportion of phosphorus—in one case as high as 0.162%, which would certainly have caused the plate to be rejected at this time, unless its use were dictated by distinctly dishonest conditions.

Again, a consideration of the circumstances and a study of the prevailing conditions and designs, show three of the re-

ported pipes to have been of most unusual and eccentric design, whilst two pipes collapsed owing to failure of designers to provide plates whose unit stress should be suitable for conditions well recognized at this date. Deducting those pipes whose partial or total destruction should have been provided against, there remains only four failures unexplained, and one of these might be placed if the history of the structure were known.

In view of this testimony and the most conclusive and practical evidence offered by the constant and increasing use of structural steel, there can be no question as to the fitness and adaptability of this product to the many purposes of the mechanical arts.

Continuing the consideration of this question, an interesting discussion upon the choice of materials may be found in Prof. W. D. Pence's "Stand-pipe Accidents and Failures," which, on account of its clearness and propriety, is presented here literally:

"**Relative Merits.**—In weighing the relative merits of steel and wrought iron as materials for the construction of stand-pipes, it may not be denied that each material has points of excellence possessed either in a less degree, or perhaps not at all, by the other. Judging alone from the recorded failures of the two metals in actual service, wrought iron appears preferable to steel. However, an entirely just interpretation of this record must recognize the fact that a majority of the total failures of steel stand-pipes may be traced to the use of ill-adapted or exceptionally inferior grades of that metal. With this qualification, the contrast in the records of the two materials is much reduced, if indeed it is not quite eliminated. Careful consideration of the foregoing records and facts related thereto leads to the following conclusions:

"(1) That steel plate of cheap grades is certainly a dangerous material to use in the construction of stand-pipes.

"(2) That steel plate of proper quality is a safe material for the construction of stand-pipes.

"(3) That wrought-iron plate, equivalent in quality to the usual grades of that material hitherto employed for stand-pipe construction, is a safe material for this purpose.

"The first of these conclusions is substantiated by a number of the more widely known failures of steel stand-pipes. The second is warranted by the scarcity of failures of steel stand-pipes, in whose construction proper grades of plate metal were used. The truth of the third is evidenced by the several classifications of accidents and failures.

"The decided preference for steel, which has grown so rapidly in other fields of work, applies with full force in the construction of stand-pipes, and it has now reached such a stage that exceedingly few concerns make a specialty of building wrought-iron stand-pipes. An important result of this evolution, which in the future may require a qualification of the third conclusion above stated, is thus described by a recognized authority in the field of structural tests: 'Steel for most structural purposes has so far replaced wrought iron that it is now difficult to get competition among the manufacturers of wrought iron for structural purposes. Many of the manufacturers who are still making wrought iron find that the demand is so much greater for steel—and in fact the profit better in steel—that they are not putting the care and attention to the manufacture of wrought iron that they have in the past, and it is getting every month harder and harder to obtain the best grades of wrought iron for structural purposes. There are, however, still a few concerns who are holding up their reputations and manufacturing as good wrought iron as in the past.'"

Another authority in the same field expresses the opinion that: "The quality of wrought iron is about the same as it was before the 'era of steel,' but engineers and inspectors

who have to deal with materials for structural purposes are no longer as familiar with iron as they were some time ago, or as they are with steel."

In view of the conflict of opinion indicated by the expressions above quoted, particular interest attaches to the following statement from a well-known firm of boiler-merchants, having an experience covering a period of more than half a century:

"There are very few mills to-day that have among their employees men who can make first-class iron, and by reason of the fact that orders for iron are so exceedingly rare and these men can be put at the work only at infrequent intervals, their skill has departed and they have no longer the ability to make as good iron as was made five or ten years ago.

"Whatever the present status of the question, it is pertinent to observe that the results of a very similar rivalry between steel and wrought iron in the manufacture of T rails, some years ago, tends forcibly to confirm the belief that the quality of the superseded metal must decline sooner or later in the case under consideration. Such deterioration having taken place, it seems quite certain that wrought iron could show no superiority over steel in open competition, and, as remarked in discussing this subject at the conclusion of the original record of accidents, it seems altogether probable that the favorable showing of wrought iron indicated by the record of stand-pipe failures would soon be forfeited were the extensive use of wrought iron for this purpose to be suddenly resumed without a corresponding restoration of the former qualities of that metal. Fortunately, the few firms that have adhered loyally to the use of wrought-iron and have built most of the large wrought-iron stand-pipes during the period of alleged retrogression, seem to have recognized the impor-

tance of using good grades of that metal, so that the decline in safety, above suggested, has probably not begun.

"Very naturally the reduced cost of steel, attended by a growing confidence in its uniformity and high quality when demanded, has led to a decided preference for that metal. That this preference will not be modified under present conditions seems very certain, but this fact will not, and very properly should not, prevent the use of wrought iron of appropriate grades when preferred. Since little assurance of excellence is to be found in the mere names steel or wrought iron, the really vital consideration is not so much which metal as what grade of the chosen metal."

Upon a subject where there is room for so wide an expression of individual opinion, and in view of the conservative tendency which bids the manufacturer as well as the engineer "Be not the first by whom the new is tried," there is little wonder at the following expression from one of the most long-established and eminently reliable and respectable metal workers upon the use of steel or iron plate in stand-pipe construction:

"We do consider iron plates more uniform in composition and better adapted for stand-pipe construction, regardless of question of cost, than steel plates of the standard chemical and physical properties, as we are able to obtain those plates. The difficulty the mills rolling plates meet with is that they can not produce all plates of the quality they desire.

"Our specifications for a stand-pipe iron plate are merely that the plate shall be double refined and fibrous in nature, not crystallized in its composition, 48,000 to 50,000 pounds tensile strength, and made from such mixture of pig iron as we know will unite in making a strong plate. We have used one mixture of pig iron, comprising three different grades of pig, for a period of twenty years in stand-pipe plates, and there never has been a failure of one plate of this material. It

may be an interesting fact for you to know that every stand-pipe which has mysteriously broken or burst, has been built of steel plates. (Statement not substantiated by facts.)

"We have no specifications of our own for steel plates, but have adopted in our use either the specifications adopted as standard by the American Rolling Mill Association, or the specifications adopted by the American Boiler Makers' Association, either of which we regard as good as can be obtained. . . . We would hesitate very much before using steel rivets in stand-pipe work. While the steel makers have made great progress and improved very much in the manufacture of steel plate, they have not met with equal success in manufacturing a rivet steel.

"The difference between the United States Naval Department and the Carnegie Company in reference to ship-plates made for the department, and to be used at Newport News, is a fair illustration of the inability of plate makers to make a uniform, homogeneous grade of steel plate in every case. If you read up in the matter, you will recall that the plates were made under strict specifications as to the physical and chemical requirements, and that every stage in the process of their manufacture was watched by experts, both on the part of the Government and on the part of the manufacturer, and yet when the plates were finished and shipped to Newport News, the ship-builders and the experts watching the construction of the work, discovered that many plates cracked. The matter was referred to a commission and it was agreed that in view of all the facts, and allowing for the inability to control the product of a steel mill, the Government could not condemn all the plates delivered, neither could they accept all, but that the use of plates would depend entirely upon the result of the shop-work at Newport News."

The foregoing having to do principally with the relative utility of the two metals and regardless of commercial con-

siderations, and as these last are governing factors in this practical age, a comparison is certainly not complete without considering market values or intrinsic worth of the two metals.

One of a set of specifications calling for proposals for wrought-iron stand-pipe construction was issued in October, 1897, the dimensions of the pipe being 15 ft. by 110 ft., the metal to conform to the following requirements:

"The material of which the stand-pipe shall be built shall be a good, sound, rolled plate, having a tensile strength of not less than forty-eight (48,000) thousand pounds per square inch of section; elastic limit, twenty-four (24,000) thousand pounds; elongation not less than 15% in a full section of test-piece 8 in. long, and on examination show no sign of inferior workmanship. Each plate shall be stamped with the name of the manufacturer and its tensile strength." The shop to whom the award was made furnished at the same time an alternate proposal for steel plate under the following manufacturers' guarantee:

Steel plate	$\frac{3}{16}$ in.	to	$\frac{1}{4}$ in.	T. S. 60,000	to 66,000	lbs.	per	sq.	in.
" "	$\frac{1}{2}$ "	"	$\frac{5}{16}$ "	T. S. 54,000	" 58,000	"	"	"	"
" "	$\frac{5}{16}$ "	"	$\frac{3}{8}$ "	T. S. 56,000	" 60,000	"	"	"	"
" "	$\frac{7}{16}$ "	and upward		58,000	" 64,000	"	"	"	"

Elastic limit more than $\frac{1}{2}$ T. S.

Elongation, 8 in. section (at least), 20% for all plates over $\frac{3}{8}$ in. thick.

Reduction of area, at least 50%.

The market prices of the two metals at the date of these proposals were as follows f. o. b. cars at mills:

Steel plate, $1.05 per 100 lbs.

Iron plate $1.40 " " "

Iron rivets 50 cts. per 100 more than steel.

The estimated weights of the stand-pipe material were as follows:

For iron, weight of plates and angles 81,600 lbs.
For steel, weight of plates and angles 85,680 lbs.

[NOTE.—Increased weight approximates an additional weight of 5% of steel over iron of like dimensions.]

Estimated amount of rivets, 4,600 lbs., including waste allowance.

The estimated cost of the superstructure, therefore, would be as follows:

81,600 lbs. iron plates at $1.40	$1142.40
85,680 lbs. steel plates " 1.05	899.64
Difference in favor of steel	$ 242.76

A comparison of the relative tensile strength of the two metals shows an advantage of about 22% in favor of steel, and had steel plate been selected, allowing for the increase of strength, the thickness might have been so reduced as to have permitted a reduction of 18,850 lbs., at the market price, effecting a further saving of $197.92, or a total saving of $440.68 had steel plate instead of wrought iron been used.

Comparative Cost.—In citing this particular 145,000-gal. stand-pipe for the purpose of arriving at conclusions as to relative cost of two possible metals, it may be urged that a higher grade of steel should have been insisted upon in order to make the comparison possible; however this may be, there can be no controversion of the fact that in equivalent metals the greater strength in proportion to volume and weight, gives steel a clear preference of something like 20% as applied to ruling prices. Such reasons have led to an almost universal demand for steel as a structural metal, and its choice may be conceded. This preference having been allowed, the particular grade of steel best adapted to constructive purposes must receive consideration.

It has been explained that structural steel is the product

of two processes, the Bessemer and open-hearth, either acid or basic.

At present there are no limitations fixed by the manufacturers' standard specifications in the matter of process of manufacture, one of the initial clauses of these specifications being "Steel may be made by either the open-hearth or Bessemer process," and no notice of the further refinement possibly resulting from the character of the furnace-lining is taken; notwithstanding this, each process of manufacture has its ardent advocates.

Comparative Homogeneity and Strength of Bessemer and Open-hearth Steels.—The Bessemer or converter process attaining its highest commercial development when operating upon a grand scale and in supplying an immense output, it is questionable whether such conditions are as favorable for scientific and exact production of steel as the less extensive furnace or open-hearth system, and where, at any period of evolution, tests may be made with regularity and certainty, and the process discontinued at the precise moment deemed most suitable.

In addition to the requirements of the manufacturers' standard specifications, the American Boiler Association demands "homogeneous" metal. If the initial metal is low in phosphorus and sulphur, the finished product may be sufficiently uniform for all practical purposes, but entire and absolute homogeneity and absence of segregation is at this time unattainable, but from the fact that in the acid open-hearth process the phosphoric and sulphuric components of the charge remain unaffected during the process of evolution, it is possible that this system of manufacture should be given a preference. This reasoning applies with equal force to the favor shown by some engineers toward the acid rather than the basic method of production, a definite allowance of some two or three per cent. sometimes being permitted, the idea being

that assurance shall be made doubly sure. It would seem that if this difference is to be recognized, the acid metal should alone be considered, except at a different commercial value, in the choice of structural steel. It is interesting to note, however, that the British Royal Navy has endorsed the following report: "With converter steel, riveted samples have given less average strength, greater variation in strength, and much more irregularity in modes of fracture than similar samples of open-hearth steel. The basic open-hearth metal has proven to be as good as that made on the acid hearth, and after full investigation, it will be used by the Admiralty in ship plates and boiler tubes on an equal footing."

In "Manufacture and Properties of Structural Steel," the author has this to say of the two processes of steel making: "My own experience leads me to think that Bessemer steel requires more work for the attainment of a proper structure than open-hearth metal, so that a thick bar is more apt to have a coarse crystalline fracture. This may be ascribed in any particular case to improper treatment, but if it is true that open-hearth metal would not be injured under a similar exposure, then it is proven that there is a difference between the metals, and if this be acknowledged, then there is no necessity for further argument.

"It is true that Bessemer metal has been used for rails, and that these are exposed to great stress and shock, but it is also true that a large number of rails break in service, and that the use of ordinary steel rail for bridges was long ago given up as dangerous. Moreover it is quite probable that the number of broken rails would be considerably reduced if they were made of open-hearth steel. It is acknowledged that the case is not yet closed, but until the foregoing statements are controverted by direct and positive evidence, the only safe way for the engineer is to prescribe that only open-hearth metal shall be used in all structures like railroad-bridges, where the steel is

under constant shock, and where life and death are in the balance. In this connection it should be stated that the method by which the steel is made cannot be discovered by ordinary chemical analysis. Certain experiments indicate that there is a difference between Bessemer and open-hearth steel in the character of the occluded gases, but this system of analysis is never resorted to in practice, and no provision is made for it in laboratories. Moreover it is doubtful if any expert would risk his reputation by asserting positively, from any such evidence, that a certain steel was made by either one or the other process. Consequently, when open-hearth metal is specified, a careful watch should be kept in the steel-works that there is no substitution of the inferior metal."

Many such honest but possibly biased arguments, controverting Mr. Campbell's opinions, might be inserted, but the tendency would be to lead us back to our starting-point, and it is possibly best to conclude with the following clear and unprejudiced, if not entirely scientific, statement of the case by a reputable trades journal:

Suitable Grades for Structural Work.—"The terms 'Bessemer' and 'open-hearth' steels have reference to methods or processes, and not necessarily to qualities. If a good quality of pig iron is made into steel by either the Bessemer or open-hearth process, it would be found that the latter was softer and more uniform under the stress of severe usage. But Bessemer steel made of good iron is better than open-hearth steel made of a cheap and inferior material. Therefore the Bessemer 'tank' steel of some manufacturers will run better than the open-hearth 'flange' steel of other makers. The name don't make the quality."

The preponderence of testimony and evidence seems to point to open-hearth metal as preferable for stand-pipe construction, but after having specified this, it is of the utmost importance to see, not only that it is furnished, but that the char-

acter of the finished product is of a suitable grade, whose chemical and physical properties having been specified, will be conscientiously made to meet the requirements. This advances two important subjects: first, What chemical and physical requirements are deemed most suitable for stand-pipe work ? and, having determined this, How can certainty in obtaining what is considered requisite be secured ?

The temperature at which steel is finished, depending obviously upon the mass being worked, has been shown to exert a marked effect upon its physical properties, and to such an extent that concessions are allowed amounting, as will be observed from the manufacturers' standard, to 10,000 pounds to cover the various widths and thicknesses of sections. There seems to be an increasing tendency to test each separate thickness, and in view of the fact that tests made from the same melt but upon different thicknesses of metal, finished at different temperatures show great variability in tensile strength, the practice seems commendable. Considering the physical characteristics of a good structural steel, authorities agree that the metal should be soft, tough, and ductile; disputing, however, as to the exact limits and variation in tensile strength. In this connection Mr. Campbell says:

"The tendency in the first epoch of steel structures was toward a hard alloy, but the later practice has been a continual progress toward toughness. There was a halt in this movement at a tensile strength of 60,000 pounds, not entirely on account of any magic virtue in the figure, but because the ordinary mild steels gave that result, and a much higher price was charged for a softer metal. The conditions to-day are somewhat different, for the reduced cost of low-phosphorus pig iron, and the introduction of the basic-hearth, have altered the economic situation.

"A steel with a tensile strength of 50,000 to 58,000 pounds per square inch is a most attractive material, possess-

ing all the good characteristics of wrought iron, with greater strength and toughness, and it seems probable that it will be extensively used in the future."

According to Campbell, the German specifications in most general use call for the following physical conditions:

"For rivets: Ultimate strength from 51,200 to 59,700 pounds per square inch; elongation, 22 per cent. in eight inches.

"For other structural material: Lengthwise tests, ultimate strength from 52,600 to 62,600 pounds per square inch; elongation, 20 per cent. in eight inches.

"Crosswise tests: Ultimate strength from 51,200 to 64,000 pounds per square inch; elongation, 17 per cent. in eight inches."

Commenting upon these requirements, Mr. Campbell says: "It is safe to say that if American engineers were satisfied with the German standards, there would not be one rejection for deficient ductility where there are twenty under our more rigid requirements; and if they would be content with a steel having an ultimate strength between 52,000 and 62,000 pounds per square inch, there would not be one-fifth the number of heats discarded for being outside of the tensile limits. The bearing of these facts upon the cost of the material is self-evident.

"I do not advocate any sacrifice of strength to economy, but I would impress upon the American engineers that this soft metal is eminently suited to structural work, while by maintaining their present chemical limitations and their requirements concerning ductility, they will be assured of a material which is equal in quality to any produced in the world."

In a recent publication, one of the largest manufacturers of structural steel records his conclusions as follows:

"The strength of structural steel depends largely on the

amount of the constituent elements that are associated with the iron, and each of which affect more or less the hardness and strength of the material.

"The principal of these are carbon, manganese, silicon, phosphorus, and sulphur, the first-named being purposely retained as useful or necessary, the others being rejected, as far as practicable, as objectionable when in excess of certain minute proportions.

"The grade and character of the steel is usually known by the percentage of contained carbon. Steel used in structures usually varies in tensile strength from 55,000 to 70,000 lbs. per square inch of section, or from .10 to .25 per cent. of carbon.

"The following table exhibits the physical characteristics of open-hearth basic steel of the various grades, the results derived from an extensive series of tests indicating the tendency of a total average of the composition hereafter described to approximate to the figures given in the table.

"The predominant elements other than carbon averaged throughout the series as follows: manganese, .40; phosphorus, .04; sulphur, .05 per cent. Any increase of these elements is attended with an increase of tensile strength and reduced ductility, and *vice versa*. The tensile strength of the steel is also affected to some extent by the temperature at which it is finished, and the rate of cooling; these influences being more apparent in the grades containing highest carbon. Therefore the values given have only a general significance, and the results of individual tests may vary widely above or below the figures in the table.

"For Bessemer or open-hearth acid process steel, the tensile strength will ordinarily be greater for the same percentage of carbon given in this table, for the reason that the proportions of phosphorus and sulphur, and sometimes manganese, are usually higher than in open-hearth basic steel, each

of these elements contributing to strength and hardness in the steel."

OPEN-HEARTH BASIC STEEL.

Percentage of Carbon.	Tensile Strength in Pounds per sq. in.		Ductility.	
	Ultimate Strength.	Elastic Limit.	Stretch in 8 inches.	Reduction of Fractured Area.
.08	54,000	32,500	32 per cent.	60 per cent.
.09	54,800	33,000	31 " "	58 " "
.10	55,700	33,500	31 " "	57 " "
.11	56,500	34,000	30 " "	56 " "
.12	57,400	34,500	30 " "	55 " "
.13	58,200	35,000	29 " "	54 " "
.14	59,100	35,500	29 " "	53 " "
.15	60,000	36,000	28 " "	52 " "
.16	60,800	36,500	28 " "	51 " "
.17	61,600	37,000	27 " "	50 " "
.18	62,500	37,500	27 " "	49 " "
.19	63,300	38,000	26 " "	48 " "
.20	64,200	38,500	26 " "	47 " "
.21	65,000	39,000	25 " "	46 " "
.22	65,800	39,500	25 " "	45 " "
.23	66,600	40,000	24 " "	44 " "
.24	67,400	40,500	24 " "	43 " "
.25	68,200	41,000	23 " "	42 " "

"**Distinguishing Terms.**—For convenient distinguishing terms, it is customary to classify steel in three grades; 'mild or soft,' 'medium,' and 'hard,' and although the several grades blend into each other, so that no line of distinction exists, in a general sense the grades below .15 per cent. carbon may be considered as 'soft' steel; from .15 to .30 per cent. carbon as 'medium'; and above that, 'hard' steel. Each grade has its own advantages for the particular purpose to which it is adapted. The soft steel is well adapted for boiler-plate and similar uses, where its high ductility is advantageous. The medium grades are used for general structural purposes, while harder steel is especially adapted for axles and shafts, and

any service where good wearing surfaces are desired. Mild steel has superior welding properties as compared with hard steel, and will endure higher heat without injury. Steel below .10 per cent. carbon should be capable of doubling flat without fracture after being chilled from a red heat in cold water. Steel of .15 per cent. carbon will occasionally submit to the same treatment, but will usually bend around a curve whose radius is equal to the thickness of the specimen; about 90 per cent. of specimens stand the latter bending-test without fracture. As the steel becomes harder, its ability to endure this bending-test becomes more exceptional, and when the carbon ratio becomes .20 per cent., little over 25 per cent. of specimens will stand the last-described bending-test. Steel having about .40 per cent. carbon will usually harden sufficiently to cut soft iron and maintain an edge."

The classification of steel seems to the average layman a little arbitrary. As shown in the preceding quotation, "For convenient distinguishing terms, it is customary to classify steel in three grades, etc." The classification according to the manufacturers' standard specifications is that "Steel shall be of four grades: 'extra soft,' 'fire-box,' 'flange or boiler,' and 'boiler-rivet' steel. Commercially, and as quoted in the trades papers, the classification is as follows: 'tank,' 'shell,' 'flange,' 'ordinary fire-box,' and 'locomotive fire-box.'"

In reply to an inquiry as to the average physical and chemical properties of each of the commercial grades, one of the largest testing-laboratories in the United States writes as follows: "While we, of course, keep records of all tests made by us, they are not tabulated nor averaged. We doubtless have on record several hundred thousand tests of all grades of material made from nearly all the different steel works in the country. We can, however, give you approximately what the different grades of steel run, as follows:

"MEDIUM STEEL (TANK).

Tensile strength........................60,000 to 68,000 lbs. per sq. in.
Elastic limit, one-half the ultimate strength.
Elongation.................................. 20 to 23%
Reduction of area............................ 40 " 45%

Chemical requirements for phosphorus and sulphur same as for "soft steel."

"SOFT STEEL (SHELL).

Tensile strength........................54,000 to 62,000 lbs. per sq. in.
Elastic limit, one-half the ultimate strength.
Elongation.................................. 25%
Reduction of area............................ 50%
If acid open-hearth steel: phosphorus under...... .085%
" " " sulphur under.......... .065%
If basic open-heath steel: phosphorus under...... .035%
" " " sulphur under......... .04%

"FLANGE STEEL.

Ultimate tensile strength...............54,000 to 62,000 lbs. per sq. in.
Elastic limit, not less than..................... 33,000 lbs.
Elongation.................................. 27%
Reduction of area............................ 50%
If acid open-hearth steel:
Phosphorus not more than................ .065%
Sulphur not more than.................... .05%
If basic open-hearth steel:
Phosphorus not more than .... .035%
Sulphur not more than .... .035%

"FIRE-BOX STEEL.

"To be made of acid open-hearth steel of the following strength:

Ultimate tensile strength...............56,000 to 64,000 lbs. per sq. in.
Elastic limit.................................. 33,000 lbs.
Elongation.................................. 28%
Reduction of area............................ 56%
Phosphorus.................................. .035%
Sulphur...................................... .035%

"LOCOMOTIVE FIRE-BOX STEEL.

[NOTE.—Specifications of Baldwin Locomotive Works.]

Tensile strength, 55,000 to 65,000 lbs. per sq. in.
Elongation, 20 to 25 per cent.
Carbon, .15 to .25 per cent.
Phosphorus, not over .03 per cent.
Manganese, not over .45 per cent.
Silicon, not over .03 per cent.
Sulphur, not over .035 per cent.

All plate to be manufactured by the open-hearth process.

"RIVET STEEL.

Tensile strength..........................50,000 to	60,000 lbs. per sq. in.
Elastic limit, one-half the ultimate strength.	
Elongation.............................25 to	28%
Reduction of area.........................50 to	55%
If acid open-hearth steel:	
Phosphorus not more than..............	.075%
Sulphur not more than....................	.06%
If basic open-hearth steel:	
Phosphorus not more than................	.035%
Sulphur not more than..................	.04%

"BOILER-RIVET STEEL.

"Same as rivet steel, except that a lower percentage of sulphur and phosphorus should be asked for, and also a slightly greater elongation and reduction."

Owing to the comparatively small quantities of rivets required in stand-pipe construction, tests for rivet-rod metal are hardly practicable, and therefore specifications governing same being useless, it would seem that the practical method of securing a suitable grade of rivet metal is to purchase by the keg of manufacturers who have a standing reputation as rivet makers, and for this certain field-tests should be required.

Specifications.—In discussing the suitability of the several grades of steel for stand-pipe construction work, Prof. Pence

has this to say: "The usual market grades of steel plate may be described as follows: Tank steel is the cheapest grade. Its low price is due primarily to the grade of stock used, giving a metal with high percentages of the detrimental elements, even without the careless manipulation which cheap work is so apt to receive. The quality of the tank steel produced by a few makers is sometimes quite good, but experience has shown it to lack uniformity, and good authorities generally agree in condemning its use in important structures. While it may display the physical excellence of the best grades of steel, 'it is apt to be hard and brittle, and should never be used in any part of a stand-pipe.' It is believed by some that a fruitful cause for the treachery of tank steel is to be found in the practice of selling under that classification steel plate which has been rejected from higher grades. It is common to find merely the tensile strength of this grade of steel specified, '60,000 T. S.' being the usual requirement.

"Shell steel is the next better grade. Its greater excellence and enhanced cost are due to the use of more care in selecting the stock and in perfecting the chemical nature of the finished product. Shell steel is used in ordinary boiler-construction, and many stand-pipes have been built from it. It is, of course, preferable to tank steel, but the best practice demands a better grade for high quality boiler and stand-pipe construction. . . . Flange steel, the next grade above shell steel, is distinguished by its uniformity, high ductility, and usually low tensile strength. It is the grade of steel plate adopted in the best practice for the construction of steam-boilers and stand-pipes. . . . Ordinary fire-box and locomotive fire-box are still higher grades of steel boiler-plate, possessing special properties which fit them for the uses indicated by their trade designations."

The matter of cost naturally has a distinct influence upon the selection of grades of materials to be used in stand-pipe

construction, and a comparison is therefore of interest. In July of the present year (1900), a large manufacturer of boilers and stand-pipes writes as follows:

"In regard to the price of steel plates, would advise

Tank steel, under $\frac{3}{16}$ in. at mill	$1.15
" " above $\frac{1}{4}$ in. at mill	1.10
Shell steel	1.20
Flange steel	1.25
Fire-box steel	1.30 to 2.85
Rivets	1.80

In addition to the chemical and physical specifications for fixing the requirements for different grades of steel, it is considered good practice to stipulate certain bending and drift tests, depending upon the nature of the work for which the steel will be used. The Testing Laboratory, before quoted, writes in this connection, "These tests frequently reject material more than other requirements, as they more clearly show whether the material will stand the strain for which it is intended."

The specifications for plate suggested by Prof. Pence for stand-pipe material is as follows: "*Material.*—The material composing the stand-pipe shall be soft, open-hearth steel, containing not more than 0.06% phosphorus, and having an ultimate tensile strength of not less than 54,000, nor more than 62,000, lbs. per sq. inch; an elastic limit not less than one-half the ultimate strength, an elongation of not less than 26% in 8 inches, and a reduction of area of not less than 50% at fracture, which shall be silky in character. Before or after being heated to a cherry red and quenched with water at 80 deg. F., the steel shall admit of bending while cold, flat upon itself, without sign of fracture on the outside of the bent portion."

The requirements above are the result of wide investigation by Prof. Pence, and plate filling these specifications

would certainly prove a suitable material, whilst the stipulations are not so severe as to appear too arbitrary or such that there should be any difficulty upon the part of the manufacturer in filling the order, hence the market quotation upon such plate should be sufficiently reasonable as to permit of its use for such structures.

Practically the steel called for by Prof. Pence is a "flange steel," worth, according to the quotations above cited, $1.25 per 100 lbs. f. o. b. at mill. One of the best authorities in the United States writes as follows regarding structural steel for stand-pipe work:

"In the matter of stand-pipe construction, the quality of the steel depends a good deal on the size of the stand-pipe. That is, on the thickness and size of the plates which you are to use. Also whether you are going to drill and ream the material. Roughly speaking, the specifications should be about as follows:"

"Soft open-hearth steel; to be either acid or basic; tensile strength, 54,000 to 62,000 lbs.; elastic limit not less than 33,000; elongation, 26%; reduction of area, 50%; sulphur, if acid open-hearth steel, less than .06%; phosphorus less than .075%. If basic open-hearth steel, phosphorus to be under .035 and sulphur under .035%. Bend tests should be made on strips about 1½ in. wide, planed parallel, and then should be bent 180 degrees flat upon themselves without showing sign of fracture on either the convex or concave side of the curve. This test should be carefully carried out on each plate. Certain drift tests should also be made; that is, a hole 15-16 in. in diameter, or whatever size the rivet-hole is, should be drifted to twice its size without cracking or injuring the plate."

This authority practically agrees with the conclusions ascribed to Prof. Pence as to the quality of steel suitable for stand-pipe work. As has been shown, the thickness of plate affects the physical properties, and should therefore, it appears

to the author, be considered in the preparation of a set of specifications. In this connection, and quoting from the "Manufacture and Properties of Structural Steel:" "The effects caused by variations in rolling temperatures appear in their most marked degree in the comparison of plates of different gauges. It is not customary to test the same heat in several sizes, but by long experience the manufacturer is able to judge the relative properties of each thickness. The heads of two widely known plate mills have given me their estimate that, taking one-half inch as a basis, there will be the following changes in the physical properties for every increase of one quarter of an inch in thickness:

(1) A decrease in ultimate strength of 1000 pounds per square inch.

(2) A decrease in elongation of one per cent., when measured in an 8 in. parallel section.

(3) A decrease in reduction of area of two per cent.

It is therefore plain that in writing specifications some allowance must be made for these conditions, since a requirement which is perfectly proper for a three-eighths inch plate will be unreasonable for a plate of one and a half inches.

"Moreover the effect is cumulative, since a hard steel must be used in making the thick plate, and this will tend to lessen the difficulty rather than make up for the reduction caused by the larger section. In plates below three-eighths of an inch in thickness it is also necessary to make allowances, since it is almost impossible to finish them at a high temperature, and the test will give a high ultimate strength and a low ductility."

Whilst it may appear unnecessary to exact as a prerequisite the percentage of permissible alloys, other, perhaps, than phosphorus and sulphur, it may not be amiss to include in the specifications, certain requirements as to silicon and manganese.

In the "Manufacture and Properties of Structural Steel"

appears a table compiled from a number of tests of groups of specimens from both acid and basic manufacture, and from this table, two groups of .109 % carbon steel show the other elements as follows:

(1) Silicon .008; Manganese, .310; Sulphur, .036; Phosphorus, .066
(2) " .007; " .380; " .048; " .082
Ultimate strength of specimen No. 1 (acid) .... 57,310 lbs.
" " " " No. 2 (basic) .... 57,430 "

According to table showing graduations of steels in relation to their percentages of carbon, it will be seen that this steel will grade as "soft"; ultimate strength, 56,500; elastic limit, 34,000 lbs.; stretch in 8 in., 30%; reduction of fractured area, 56%.

It is impossible at this time to reconcile all conclusions, and theoretical and scientific considerations must be moulded more or less to fit commercial standards, which have been largely set by the Association of American Steel Manufacturers, whose standard specifications are the result of much careful consideration and study.

Deviations from these regulation specifications will be found to entail additional expense to the consumer, possibly not warranted by assumed theoretical conditions, and therefore, in the matter of physical test of steel required, the wording of the specifications "to conform to the standard specifications of the Association of American Steel Manufacturers," would undoubtedly cover the general physical requirements for a serviceable steel which should be "soft," 52,000 to 62,000 lbs. tensile strength per square inch.

In the matter of the chemical specifications, this properly comes within the province of the engineer, and the following is suggested:

CHEMICAL SPECIFICATIONS.

The plate metal to be used in stand-pipe construction shall be the product of some well-established and reputable

mill employing the "open-hearth process of manufacture," a preference being given to acid furnace-lining methods.

The chemical qualifications for this metal shall be such as to ensure the reduction of the metalloids to the following limiting maximum percentages in the finished product:

Phosphorus, .07; Sulphur, .05; Manganese, .60; Silicon, .04.

Drillings for chemical analysis may be taken either from test-piece or finished product, and if required, each of the elements may be ordered determined.

The simple tests of bending and drifting should be inserted into the specifications for structural metal. It should be provided that from any melt or number of melts, test-specimens, as strips, might be cut from the plate. Such strips should be about $1\frac{1}{2}$ inches in width, should be planed parallel, and, when bent 180 degrees upon itself, either hot or cold, should fracture appear upon either the concave or convex surfaces of the curve, the melt may be subject to rejection. Rejections should also be provided for if the material will not stand, without injury, drifting a hole in test pieces to twice the original diameter. Such holes are ordinarily about $\frac{15}{16}$ in.

Inspection.—That there may be no uncertainty or disappointment as to results, it is necessary not only that the constructive engineer shall know what to specify in ordering materials, but he must be reasonably sure that he is getting what he requires. No field-inspection or cursory examination can be relied upon to reveal departures from the specifications and fatal defects, and absolute certainty as to results can only be secured through a close, systematic inspection during the process of manufacture from the raw material to the finished structure; it is obvious, therefore, that such careful attention to details requires the constant presence of a skilled inspector at the mill, the shop, and in the field. A knowledge of and the ability to conduct the necessary series of chemical and

physical tests is rarely possessed by the designing and constructing engineer, even though it were possible for him to give his personal attention to these details, hence, very properly, such work is now entrusted to an assistant making a specialty of such work, or most usually to some reputable inspection-bureau, the outgrowth of this condition.

The necessity for, and extent of, this practice is clearly explained in a recent paper entitled "Shop and Mill Inspection," by Mr. W. O. Henderer, read before the Civil Engineers' Club of Cleveland, and from which the following is quoted:

"There was a time when one man could comfortably attend to such duties himself, and personally follow the progress of the material in all its various processes. The shops and mills at which iron was manufactured, and where the finished parts of structures were produced, were often one and the same; or, if not, the processes followed each other in such rotation that one man could get from mill to shop and keep proper consecutive track of the work. But the industry has of late years grown to such enormous proportions and has extended over such a large area that it is impossible for one man to properly inspect the work in all its stages. Bridge companies now have a number of mills from which to order the material necessary for their work. They are likely to have plates from one mill, beams and channels from another, and other shapes from still a third; and the mills are often great distances apart. Frequently, too, the shop is at work on some portions of a contract while the mills are still furnishing materials. It is manifestly out of the question for any one man to thoroughly inspect work at all these places at one time. He must have assistance in some way.

"Men who have become expert and experienced in this sort of work have made inspection their particular business, performing this service at a compensation based on the ton-

nage in the work, instead of entering the service of the engineer or architect in charge at a salary. Such men, as they found it impossible to economically perform their duties personally on account of the excessive expenses of travelling about, adopted the method of reciprocating among themselves, an inspector in Pittsburg undertaking to do the mill-inspection on one piece of work for another located in Philadelphia, while the latter attended to shop-inspection at shops in his vicinity for the former. Naturally, from such alliances among inspectors, there has resulted the formation of inspection-bureaus or companies. Such companies employ men permanently at the various mills and shops, and maintain extensive general offices, at which the clerical work of copying and forwarding reports and tests, progress of work, etc., is performed. By securing large quantities of inspection work they are able to keep good men at all the localities necessary, maintaining a perfect system of effective inspection and giving their clients regular reports of the quality of material and workmanship, the progress of the work, and information as to tests, shipments, etc., which, when completed, comprises an accurate record of the structure in question, and surety that it is built as it should be. . . . The employment of competent inspection-bureaus becomes more and more general as the iron and steel industry increases in volume, and competition amongst the manufacturers grows keener. Men are realizing more and more forcibly the necessity for such services in order to secure good results. The day when people thought that because a bridge was built of iron it would stand indefinitely is past and gone. Men are finding that there is good and bad iron and steel, and that there is a great difference between them—often the difference between success and failure, between a strong, stiff, and durable structure and an accident costing human life—that it pays to spend the small added cost to insure the use of good material and to detect and exclude the bad.

"It is remarkable that so many fail to see that specifications and inspection must always go hand in hand; that neither can confer the benefits it should without the other. Most people realize that if no specifications are stated to indicate the nature and quality of the structure desired, the manufacturer cannot be blamed if the structure does not meet the expectations of the purchaser. But often little thought is given to the second part of the purchaser's duty, that of inspection. It is not recognized as a duty owed by every purchaser for his own protection and safety, and to secure benefits from a carefully compiled specification. When the millennium is reached, when it may be reasonably expected that every man's work will be perfect and each one's labor as valuable as that of his fellows, then there will be no difference between good and bad, no possibility of errors or mistakes or dishonesty. When that time arrives there will be no further use for either specifications or inspection, and many a busy man will loose his job. But until that time there will be varying grades in the quality of materials and workmanship, and the necessity for specifying the grade desired on any piece of work will remain.

"And just so long as there is any cause or reason for specifications, just so long will the inspector be needed to see that the specifications are carried out."

Concerning the character of the inspection and cost for same, Mr. Henderer continues: "There are a few inspection-bureaus who are striving for the improvement of inspection services, through the establishment of carefully devised systems for the thorough handling of the work and the employment of only experienced and thoroughly reliable men. Such companies can and do give the quality of service that makes inspection thoroughly valuable. But they have thus far found themselves seriously handicapped by the many irresponsible inspectors who undertake work at ridiculously low

prices without any idea of doing it as it should be done. Engineers and architects are not a little to blame for this state of things, since too many of them fail to consider the inspection service as one having degrees of quality. They have become accustomed to consider that all inspection is the same, and to require that each inspector who makes application for their work shall submit his prices in competition with any one else who may be an applicant, and then employ the man with the lowest price without taking the trouble to properly investigate the comparative facilities or reputations of the applicants.

"It cannot be expected that the best results of inspection will be gained by crowding the price for such services down to the lowest possible figure. There is a limit below which good inspection cannot be performed. The only way in which an engineer can get the full benefit that inspection can confer is to determine at the outset to pay a fair price for that service, and then, before appointing an inspecting firm, to look carefully into the reputations of the different inspecting companies available, by references to other engineers and to pieces of work that have been inspected by them.

"Thorough and complete inspection of iron and steel structural material should generally be worth one dollar per net ton of shop shipping-weights. At times, and under especially favorable conditions as regards the location of the bureau's employés, it can be done for less. On some small jobs it may be more, but there is in general a chance for the inspector to make a fair living at that average price. Such inspection should include the careful comparison and checking of working plans, and complete supervision and tests by thoroughly experienced, expert, and reliable men throughout the manufacture of the material from the time it is first produced until it is shipped from the shop."

CHAPTER IV.

STRESS OR STRAIN.

"STRESS" or "strain" is the name designating the application of forces to a body in the same straight line but in opposite directions, so that the internal resistance offered by the cohesive force of the fibres or particles of which the body is composed is balanced by the opposing or exterior force or pressure.

The effect of an exterior force acting upon a body to change its shape, may be exerted as "tension," "compression," or "shear."

If the force acting upon a body has a tendency to elongate or stretch its fibres to the point of rupture by pulling them apart, this force is termed a "tensile stress."

If, on the contrary, the application of the force tends to shorten or to compress these fibres, such force is called a "compression stress," obviously "compression" and "tensile" stresses differ only as regards the direction in which the exterior force is applied or exerted upon the fibres of which the body consists.

Force applied so as to act longitudinally along any "member" of a structure through its fibres, tends either to elongate or to compress these fibres in direct proportion to the pressure exerted, and the resistance offered to this pressure by the fibres themselves is also directly proportional to the tenacity and number of the fibres of which the body is composed. as represented by its area or "cross-section."

Beside these two stresses, there is a third, called a "shear stress," and which, as its name would indicate, is the tendency

of the external force to cut in twain or to shear the fibres, and is the application of the forces in vertical planes at right angles to the fibres, or through the cross-section of the body.

The consideration and understanding of these stresses in the material and members of such structures as towers, tanks, and the like, and a knowledge of the resistance which the character of the material, its dimensions, and shape, will offer in opposition to extraneous forces is of the utmost importance.

The manner or method of the application of force to a body necessarily comprehends a principle of mechanics known as the "moment" of forces, or the tendency of a force to produce motion about a point. This is an expression representing the power produced by the force to cause motion about a point when acting through the principle of "leverage."

In the consideration of the stability of a structure or its ability to resist a sliding, horizontal motion, or a tendency to overturn about its toe, the consideration and application of the principles of "leverage," and the opposing force exerted by the natural law of gravitation, must be carefully analyzed and observed.

Moment of Forces.—The "moment" of a force is the product of the force by its leverage; thus, if the force or pressure be represented by pounds, tons, etc., and the leverage of the force, or the perpendicular or shortest distance from its "fulcrum" to the direction through which the force is acting is expressed in feet, this product is termed the "moment" of the force about the given point, and may be expressed as "foot-pounds" or "foot-tons."

If any force, as 10 pounds, 10 tons, etc., be exerted through a leverage of any number of feet, say 20, the resultant, 10 $\times$ 20 equals 200 feet-pounds or feet-tons.

The resistance which the weight of a structure, acting ver-

tically through its centre of gravity, offers to an applied force through its leverage and tending to change its position determines its "stability of position."

Equilibrium.—Forces are said to be in "equilibrium" when they equal or balance each other, each preventing the other from imparting motion to the body; so also forces, when multiplied by their respective leverages, are said to be in equilibrium when the action which each exerts maintains the body at rest, and it may be observed that the moment of forces about a point may hold each other and establish the equilibrium of the body even though the forces themselves fail to balance. Two opposing forces, or the moment of these forces, acting at the same time equally upon an unresisting body, neutralize or destroy each other, the body is at rest and equilibrium is said to exist. Should one force, or the moment of that force, exceed the other, equal parts of each force destroy each other and any excess of the one over the other is termed the "resultant" of the two forces; and the direction of this excess, or the resultant of the two forces, is exerted in a line bisecting the original angle at which the forces met, and the extent of the force exerted by this resultant is the difference between that offered by the two or more original forces, or the moment of those forces.

Resistance to Overturning.—In analyzing the stability of any structure such as a stand-pipe, the effect of the pressure exerted by the wind against the sides of the tank is to cause motion by a sliding, horizontal movement, and to produce overturning about the toe or base. This tendency is resisted by the weight of the tank itself, acting vertically through its centre of gravity and upon the area of its base. The disposition toward moving horizontally upon its base is opposed by the roughness of the parallel faces in contract, as the bottom plates of the tank and the upper face of the foundations, and is found by multiplying the perpendicular pressure by the "coefficient of friction," but as against the action of the

wind upon the sides of a stand-pipe, the vertical pressure exerted even by the weight of the empty tank over the area of its base, is usually sufficient to restrain the force exerted by the wind and to keep the structure at rest even without the customary anchorage, therefore this tendency will not be given further consideration here.

The effect which wind exerts upon cylindrical structures such as a stand-pipe has never been determined with any degree of certainty, but Trautwine has the following:

Wind Pressure.—"The relation between the velocity of wind and its pressure against an obstacle placed either at right angles to its course, or inclined to it, has not been well determined, and still less so its pressure against curved surfaces. The pressure against a large surface is probably proportionately greater than against a small one. It is generally observed to vary nearly as the square of the velocities, and when the obstacle is at right angles to its direction, the pressure in pounds per square foot of exposed surface is considered to be equal to the square of the velocity in miles per hour, divided by 200. On this basis, which is probably quite defective, the following table, as given by Smeaton, is prepared:"

Velocity in Miles Per Hour.	Velocity in Feet Per Second.	Pressure in Pounds Per Square Foot.	Remarks.
1	1.467	.005	Hardly perceptible.
2	2.933	.020	Pleasant.
3	4.400	.045	
4	5.876	.080	
5	7.33	.125	
10	14 67	.5	
12½	18.33	.781	Fresh breeze.
15	22.	1.125	
20	29.33	2.	
20	36.67	3.125	Brisk wind.
30	44.	4.5	Strong wind.
40	58.67	8.	High wind.
50	73.33	12.5	Storm.
60	88.	18.	Violent storm.
80	117.3	32.	Hurricane.
100	146.7	50.	Violent hurricane.

The assumption given above, that the pressure of the wind acting upon a semi-cylindrical surface is equal to that which would be exerted upon a flat surface, having an area equal to that of the diametral plane of the cylinder, is generally accepted as nearly correct by the best authorities, and accords with the recommendation of Rankine in Applied Mechanics.

In assuming the maximum pressure of the wind, it is considered good practice to accord it a pressure of about 30

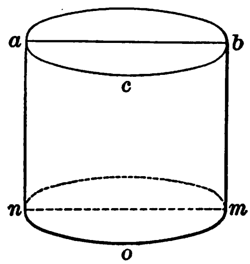


FIG. 1.—The pressure against a semi-cylindrical surface *a b c n o m* is about that against the flat surface *a b n m*.

lbs. per square foot and estimated as being exerted upon the vertical plane as projected through the centre of gravity of a cylindrical structure; thus, to estimate the maximum pressure of the wind exerted upon the semi-cylindrical sides of a stand-pipe 20 ft. in diameter and 120 ft. in height, $20 \times 120 \times 30$ lbs. equals 72,000 lbs. or 36 tons, and the moment of this force, or the pressure in tons multiplied by its leverage, or its distance from the centre of gravity about the point, is 60 ft. $\times$ 36 tons, or 2,160 ft.-tons.

The resistance offered to this overturning moment is the weight of the structure, in tons, multiplied by its leverage, or its perpendicular distance from its centre of gravity at its base to the point or toe, and as the centre of gravity of a cylinder is the centre of the circle, the leverage is therefore

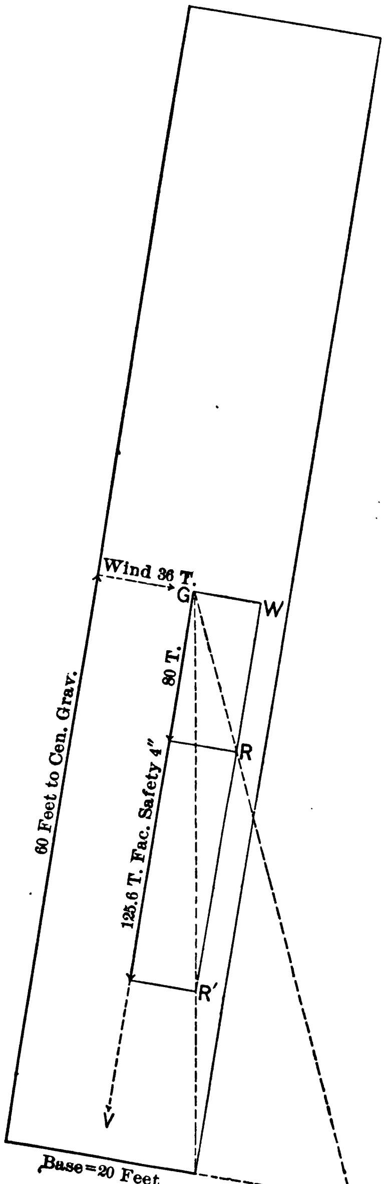

Wind 36 T.
G
W
80 T.
R
125.6 T. Fac. Safety 4"
R'
V
60 Feet to Cen. Grav.
Base=20 Feet

FIG. 2.

its radius, or in this case 10 ft., so that the moment of this force is its weight, say 80 tons, multiplied by its lever-arm, 10 ft., or 800 ft.-tons, therefore the resultant of these two moments shows an excess of 1360 ft.-tons, in amount and tendency sufficient to render the structure unstable or to cause its overturning. In order, therefore, to render such a structure stable upon its foundations, it will be necessary to provide a suitable anchorage. In order to show the instability of such a structure graphically, lay off, by scale, a figure 20×120, denoting its centre of gravity G. Draw the horizontal line GW to any convenient scale, representing the estimated force of the wind in tons. By the same scale, draw a vertical line, GV, showing the direction and amount of the vertical forces due to the weight of the structure. Complete the parallelogram of forces as shown, and the diagonal, GR, will represent the direction and extent of the combined action of the vertical and horizontal forces, and, if produced, falls without the figure or beyond its base. Here the structure can not stand.

In order to secure the equilibrium of the structure, it is evident that some form of anchorage must be provided, and we will therefore assume that eight 2-in. iron rods, of 40,000 lb. per square inch unit tensile strength, would be sufficient when firmly set in the foundations of masonry. Each rod being capable of exerting a "holding down" pressure of approximately 62.8 tons. In structures of this character, not subject to sudden jar or shock, the usual practice is to proportion the members so as to assure a working strength at least four times greater than theoretical requirements would demand, and to discount the liability of failure through possible physical defects of the materials to that extent. The "ultimate strength" of the material, when divided by the "unit stress," determines the "factor of safety," or in this case, $\frac{62.8}{4}$ equals 15.7 tons, which, multiplied by the number of rods, gives

125.6 tons, added to the actual weight of the structure, 80 tons, jointly tend to hold the tank upon its foundations. The extent and direction of these added forces can be graphically shown as before, and their resultant produced, R′, falls within the diagram.

To prove this mathematically, using the principle of moments, we will assume that the bolts are centred 11 ft. from the centre of the base of the tank, or 1 ft. beyond the external diameter of the cylinder. The weight of the tank itself, 80 tons, multiplied by its leverage, 10 ft., equals 800 ft. tons, plus the downward pressure of the anchorage, 125.6 tons, multiplied by its leverage, 11 ft., or 1381.6 ft. tons, gives a total moment of the vertical forces as 2181.6 ft. tons. Now as the pressure of the wind, acting through its leverage of 60 ft., has been shown to give a horizontal moment of 2160 ft. tons, the tank stability of position is assured and an excess of 21.6 ft. tons a variance upon the right side.

Hydrostatic Pressure.—In addition to the external pressure exerted by the wind, stand-pipes are subject to, and must be designed to resist, an internal pressure of water with which they will be filled, or to resist the "Hydrostatic Pressure." From experiment it has been found that the maximum density of water occurs at from 6 degrees to 7 degrees above freezing point, from which point its density decreases and volume increases with each degree of advancing temperature.

At the level of the sea, the approximate atmospheric pressure of 14½ lbs. per sq. in. will balance a column of water 34 ft. in height. The weight of water is approximately 62½ lbs. per cubic foot, and is usually so taken for the purpose of calculation. A cubic foot of water, in a cubical receptacle, exerts a pressure over the base of 144 sq. inches, equivalent to its weight; so then, the pressure of 62½ lbs. of water over 144 sq. inches ($\frac{62.5}{144}$) equals 0.433507 lbs.; hence, to find the pressure of any column of water, multiply the height or "head" in feet by .434; very roughly, divide the given head by 2.

Conversely, when the pressure per sq. inch is given, to find the head to which the pressure is due, $\frac{144}{62.5}$ equals 2.30677, or roughly, 2.3. The following table may be found useful:

Converting Feet-head of Water into Pressure per Square Inch.		Converting Pressure per Square Inch into Feet-head of Water.	
Feet-head.	Pounds per Square Inch.	Pounds per Square Inch.	Feet-head.
10	5.33	5	11.54
15	6.50	6	13.85
20	8.66	7	16.16
25	10.83	8	18.47
30	12.99	9	20.78
35	15.16	10	23.09
40	17.32	15	34.63
45	19.40	20	46.18
50	21.65	25	57.72
55	23.82	30	69.27
60	25.99	35	80.81
65	28.15	40	92.36
70	30.32	45	103.90
75	32.48	50	115.45
80	34.65	55	126.99
85	36.81	60	138.54
90	38.98	65	150.08
95	41.14	70	161.63
100	43.31	75	173.17
105	45.57	80	184.72
110	47.64	85	196.26
115	49.91	90	207.81
120	51.97	95	219.35
125	54.25	100	230.90
130	56.30	110	253.98
135	58.59	120	277.07
140	60.63	130	300.16
145	62.93	140	323.25
150	64.96	150	346.34

In considering the effect of the pressure due to the height or head of water, or "static head," exerted upon the interior surfaces of a cylindrical structure such as a stand-pipe, the explanation given by Trautwine is so concise and clear that it is copied here without further apology:

"In the figure, which represents a vessel full of water, the total pressure against the semi-cylindrical surface *a v e m d k*

and perpendicular to it, must be also horizontal, because the surface is vertical; but inasmuch as the surface is *curved*, this total pressure acts against it in many directions, which might be represented by an infinite number of radii drawn from *o* as a centre. But let it be required to find the horizontal pressure in lbs. in one direction only, say parallel to *o e*, or perpendicular to *a d*, which would be the force tending to tear the curved surface away from the flat sides *a b n v*, and *d c s k*, by producing fractures along the lines *a v* and *d k*, or which would tend to burst a pipe or other cylinder. In this case, multiply together the area of the vertical projection *a d k v* in sq. feet; the depth of the centre of gravity of the curved surface in ft. (which in the semi-cylinder would be half of *e m*, or of *o i*), and 62.5.

"Since the resulting pressure is resisted by the strength of the vessel along the two lines *a v* and *d k*, it is plain that each single thickness along those lines need only be sufficient to resist safely *one-half* of it; and so in the case of pipes or other cylinders, such as hooped cisterns or tanks."

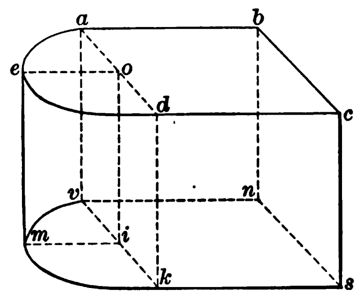


FIG. 3.

Resistance offered by Material.—From the above, it will be seen that a formula for hydrostatic pressure exerted upon the sides of a cylinder would be

$$\frac{D \times H \times 62.5}{2}, \quad \ldots \ldots \quad (1)$$

where $D =$ diameter of cylinder;

$H =$ its height in feet.

It has been shown that the pressure exerted upon the bottom of the vessel is in direct proportion to the head of water, or the area, multiplied by the head of the column in pounds.

To resist the internal hydrostatic stresses is opposed the thickness and material of the plate and its riveting in a cylindrical stand-pipe, and to proportion the opposing plate to safely resist the pressure the following factors must be known or assumed: 1st, The tensile strength of the metal; 2d, the percentage of strength of the material; 3d, a reduction of theoretical strength to allow a margin or factor of safety; and 4th, some unit of length must be adopted representing the surface pressed. The unit of length is usually taken for convenience at 12 in. In designing, 60,000 lbs. per sq. in. is generally assumed as the unit stress of the material, and allowance for the decreased value of this unit, due to punching and riveting, is made at about 33 per cent. off, or the working value of a 12 in. section is at $\frac{2}{3}$ of its original strength; reducing the ultimate strength by using a factor of safety of 4 is considered good practice for such metal structures, not subject to shock, hence the formula for proportioning the thickness of plates intended to resist such hydrostatic pressures may be given as

$$\frac{60{,}000 \times 12'' \times \frac{2}{3}}{4} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

To proportion the thickness of metal intended to resist the hydrostatic pressure exerted upon the internal surface of any cylinder, divide (1) by (2), therefore the following general expression for the thickness of metal in decimals of an inch for any given diameter of tank and any assumed height:

$$\frac{D \times H \times 62.5}{2} \div \frac{60{,}000 \times 12'' \times \frac{2}{3}}{4}$$

from the above the following original tables have been computed:

10-FT. DIAMETER CYLINDER.

Circumference, 31.4159; area, 78.5398.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure, Pounds.	Thickness, Dec. In.	Thickness, Frac. In.
10	5,890	49,087	3,124		
15	8,835	73.631	4,688		
20	11,781	98,175	6,250		
25	14,726	122,718	7,812		
30	17.671	147,262	9.374		
35	20.617	171,806	10,938		
40	23,562	196,350	12,500		
45	26,507	220,892	14,062		
50	29,452	245,437	15,624		
55	32.397	269,981	17,188		
60	35.343	294,524	18,750		
65	38,288	319,068	20.312		
70	41,233	343.611	21,876	.1823	3/16
75	44.179	368,155	23,438	.1953	3/16
80	47,124	392,699	25,000	.2083	13/64
85	50,069	417,242	26,562	.2213	7/32
90	53,014	441,786	28,124	.2344	15/64
95	55,960	466,330	29,686	.2474	15/64
100	58,905	490,874	31,250	.2604	1/4
105	61,850	515,417	32,812	.2751	9/32
110	64,795	539,961	34,374	.2864	9/32
115	67,741	564.505	35,936	.2995	19/64
120	70,686	589,048	37.500	.3125	5/16

11-FT. DIAMETER CYLINDER.

Circumference, 34.5575; area, 95.0332.

10	7,127	59,396	3,438		
15	10,691	89,093	5,156		
20	14.255	118,791	6,874		
25	17,819	148,489	8,594		
30	21,382	178,187	10,312		
35	24,946	207,885	12,032		
40	28,510	237,583	13,750		
45	32,073	267,280	15,468		
50	35,637	296,978	17 188		
55	39,201	326,676	18,906		
60	42,764	356,374	20,624		
65	46,328	386,072	22,344	.1862	3/16
70	49,892	415,770	24,062	.2005	3/16
75	53,456	445,468	25.780	.2146	7/32
80	57,020	475,165	27,500	.2292	7/32
85	60,584	504,864	29,218	.2435	15/64
90	64,147	534,562	30,936	.2578	1/4
95	67,711	564,259	32,656	.2721	9/32
100	71,275	593,957	34,374	.2864	9/32
105	74,839	623,655	36,092	.3007	19/64
110	78,402	653,353	37,812	.3151	5/16
115	81,966	683,051	39,530	.3294	21/64
120	85,530	712,749	41,250	.3438	11/32

12-FT. DIAMETER CYLINDER.

Circumference, 37.6991; area, 113.10.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure, Pounds.	Thickness, Dec. In.	Thickness, Frac. In.
10	8,483	70,687	3,750		
15	12,724	106,031	5,626		
20	16,965	141,375	7,500		
25	21,206	176,719	9,376		
30	25,448	212.063	11,250		
35	29,689	247,406	13,126		
40	33,930	282.750	15,000		
45	38,171	318,094	16,876		
50	42,413	353.438	18,750		
55	46,654	388,781	20,626		
60	50,895	424,125	22,500	.1875	3/16
65	55,136	459 469	24,376	.2031	13/16
70	59,378	494,813	26,250	.2187	7/32
75	63,619	530,156	28,126	.2335	15/64
80	67,860	565,500	30,000	.2500	1/4
85	72,101	600,844	31,876	.2656	17/64
90	76,343	636,187	33,750	.2809	9/32
95	80,584	671,531	35,626	.2969	19/64
100	84,825	706,875	37,400	.3117	5/16
105	89,066	742,219	39,376	.3281	22/64
110	93,308	777,562	41,250	.3437	11/32
115	97,549	812,906	43,126	.3594	23/64
120	101,790	848,250	45,000	.3750	3/8

13-FT. DIAMETER CYLINDER.

Circumference, 40.8407; area, 132.7323.

10	9,955	82,958	4,067		
15	14,932	124,437	6,094		
20	19,910	165,915	8,126		
25	24,887	207,394	10,156		
30	29,865	248,872	12,188		
35	34,842	290,352	14,218		
40	39,820	331,831	16,250		
45	44,797	373,310	18,282		
50	49,775	414,738	20,312		
55	54,752	456,267	22,344	.1862	3/16
60	59,730	497,746	24,374	.2031	13/64
65	64,707	539,225	26,406	.2200	7/32
70	69,684	580,704	28,438	.2369	15/64
75	74,662	622,183	30,468	.2538	1/4
80	79,639	663,661	32,500	.2708	17/64
85	84,617	705,140	34,532	.2878	9/32
90	89,594	746,619	36,562	.3046	19/64
95	94,572	788,098	38,584	.3216	5/16
100	99,549	829,577	40,626	.3384	21/64
105	104,527	871,056	42,656	.3554	11/32
110	109,504	912,535	44,688	.3724	3/8
115	114,482	954,013	46,718	.3892	25/64
120	119,459	995,492	48,750	.4062	13/32

14-FT. DIAMETER CYLINDER.

Circumference, 43.9823; area, 153.9380.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure, Pounds.	Thickness, Dec. In.	Thickness, Frac. in.
10	11,545	96,211	4,376		
15	17,318	144,317	6,562		
20	23,091	192,423	8,750		
25	28,863	240,528	10,938		
30	34,636	288,634	13,124		
35	40,408	336,739	15,312		
40	46,181	384,845	17,500		
45	51,954	432,951	19,688		
50	57,727	481,056	21,876	.1822	3/16
55	63,499	529,162	24,062	.2005	13/64
60	69,272	577,268	26,250	.2187	7/32
65	75,045	625,373	28,438	.2369	15/64
70	80,817	673,479	30,626	.2588	1/4
75	86,590	721,584	32,812	.2736	17/64
80	92,363	769,690	35,000	.2916	9/32
85	98,135	817,796	37,188	.3098	19/64
90	103,908	865,901	39,376	.3280	5/16
95	109,681	914,007	41,562	.3464	11/32
100	115,454	962,113	43,748	.3644	23/64
105	121,226	1,010,218	45,936	.3828	3/8
110	126,999	1,058,324	48,124	.4010	13/32
115	132,772	1,106,429	50,310	.4192	27/64
120	138,544	1,155,450	52,498	.4374	7/16

15-FT. DIAMETER CYLINDER.

Circumference, 47.1239; area, 176.7146.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure, Pounds.	Thickness, Dec. In.	Thickness, Frac. in.
10	13,254	110,447	4,688		
15	19,880	165,670	7,032		
20	26,507	220,893	9,374		
25	33,134	276,117	11,718		
30	39,761	331,340	14,062		
35	46,388	386,563	16,406		
40	53,014	441,786	18,750		
45	59,641	497,010	21,094		
50	66,268	552,233	23,436	.1953	3/16
55	72,895	607,456	25,780	.2146	13/64
60	79,522	662,680	28,124	.2344	15/64
65	86,148	717,903	30,468	.2538	1/4
70	92,775	773,126	32,812	.2730	9/32
75	99,402	828,350	35,156	.2930	9/32
80	106,029	883,573	37,500	.3124	5/16
85	112,656	938,796	39,844	.3320	21/64
90	119,282	994,020	42,188	.3516	11/32
95	125,909	1,049,243	44,532	.3710	3/8
100	132,536	1,104,466	46,874	.3908	25/64
105	139,163	1,159,690	49,218	.4100	13/32
110	145,789	1,214,913	51,562	.4296	27/64
115	152,416	1,270,136	53,906	.4492	29/64
120	159,043	1,325,359	56,250	.4688	15/32

16-FT. DIAMETER CYLINDER.

Circumference, 50.2655; area, 201.0619.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure. Pounds.	Thickness, Dec. In.	Thickness, Frac. In.
10	15,080	125,664	5,000		
15	22,619	188,495	7,500		
20	30,160	251,327	10,000		
25	37,699	314,159	12,500		
30	45,239	376,991	15,000		
35	52,779	439,823	17,500		
40	60,319	502,655	20,000		
45	67,858	565,486	22,500	.1875	3/16
50	75,398	628,318	25,000	.2083	13/64
55	82,938	691,150	27,500	.2291	7/32
60	90,478	753,982	30,000	.2500	1/4
65	98,018	816,814	32,500	.2708	17/64
70	105,557	879,646	35,000	.2916	9/32
75	113,097	942,478	37,500	.3124	5/16
80	120,637	1,005,309	40,000	.3332	21/64
85	128,177	1,068,141	42,500	.3540	11/32
90	135,717	1,130,973	45,000	.3750	3/8
95	143,257	1,193,805	47,500	.3960	25/64
100	150,796	1,256,637	50,000	.4166	13/32
105	158,336	1,319,469	52,500	.4374	7/16
110	165,876	1,382,300	55,000	.4584	29/64
115	173,416	1,445,132	57,500	.4792	15/32
120	180,956	1,507,964	60,000	.5000	1/2

17-FT. DIAMETER CYLINDER.

Circumference, 53.4071; area, 226.9800.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure. Pounds.	Thickness, Dec. In.	Thickness, Frac. In.
10	17,023	141,862	5,312		
15	25,535	212,794	7,968		
20	34,047	283,725	10,624		
25	42,559	354,656	13,282		
30	51,070	425,587	15,938		
35	59,582	496,519	18,584		
40	68,094	567,450	21,250		
45	76,606	638,381	23,906	.1992	3/16
50	85,117	709,312	26,562	.2213	7/32
55	93,629	780,244	29,218	.2434	1/4
60	102,141	851,175	31,874	.2656	17/64
65	110,653	922,106	34,532	.2878	9/32
70	119,164	993,037	37,188	.3098	5/16
75	127,676	1,063,969	39,844	.3320	21/64
80	136,188	1,134,900	42,500	.3544	11/32
85	144,699	1,205,831	45,156	.3762	3/8
90	153,211	1,276,762	67,812	.3984	25/64
95	161,723	1,347,694	50,468	.4206	27/64
100	170,235	1,418,625	53,124	.4426	7/16
105	178,747	1,489,556	55,782	.4648	15/32
110	187,258	1,560,488	58,438	.4874	31/64
115	195,770	1,631,419	61,094	.5090	1/2
120	204,282	1,702,350	63,750	.5278	17/32

18-FT. DIAMETER CYLINDER.

Circumference, 56.5487; area, 254.4690.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure, Pounds.	Thickness, Dec. In.	Thickness, Frac. In.
10	19,085	159,043	5,625		
15	28,628	238,565	8,438		
20	38,170	318,086	11,250		
25	47,713	397,608	14,062		
30	57,255	477,129	16,876		
35	66,798	556,651	19,686		
40	76,341	636,172	22,500	.1875	3/16
45	85,883	715,694	25,312	.2109	7/32
50	95,426	795,215	28,124	.2344	15/64
55	104,968	874,737	30,936	.2578	1/4
60	114,511	954.258	33,750	.2812	9/32
65	124,054	1,033,780	36,562	.3040	5/16
70	133,596	1,113,302	39,374	.3280	21/64
75	143,139	1,192,823	42,186	.3516	11/32
80	152,681	1,272,345	45,000	.3758	3/8
85	162,224	1,351,866	47,812	.3984	25/64
90	171,766	1,431,388	50,624	.4218	27/64
95	181,309	1,510,910	53,436	.4452	7/16
100	190,852	1,590,431	56,250	.4688	15/32
105	200,394	1,669,953	59,062	.4922	31/64
110	209,937	1,749,474	61,874	.5156	1/2
115	219,479	1,828,996	64,874	.5409	35/64
120	229,022	1,908,517	67,500	.5626	9/16

19-FT. DIAMETER CYLINDER.

Circumference, 59.6903; area, 283.5287.

10	21,265	177,205	5,968		
15	31,897	265,808	8,906		
20	42,529	354,411	11,875		
25	53,162	443,014	14,844		
30	63,794	531,616	17,812		
35	74,426	620,219	20,781		
40	85,059	708 822	23,750	.1896	3/16
45	95,691	797,424	26,718	.2226	7/32
50	106,323	886,023	29,687	.2472	15/64
55	116,956	974.630	32,656	.2721	17/64
60	127,588	1,063,232	35,625	.2969	19/64
65	138,220	1,151,835	38,594	.3216	21/64
70	148,852	1,240,438	41,562	.3463	11/32
75	159,485	1,329,041	44,532	.3711	3/8
80	170,117	1,417,644	47,500	.3958	25/64
85	180,750	1,506,246	50,468	.4205	13/32
90	191,382	1,594,849	53,437	.4453	7/16
95	202,014	1,683,452	56,406	.4700	15/32
100	212,646	1,772,054	59,375	.4948	1/2
105	223,279	1,860,657	62,344	.5195	33/64
110	233,911	1,949,260	65,312	.5443	35/64
115	244,543	2,037,862	68,281	.5690	9/16
120	255,176	2,126,465	71,250	.5937	19/32

20-FT. DIAMETER CYLINDER.

Circumference, 62.8318; area, 314.1593.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure, Pounds.	Thickness, Dec. In.	Thickness, Frac. In.
10	23,562	196,350	6,250		
15	35,343	294,524	9,375		
20	47,124	392,700	12,500		
25	58.905	490,874	15,625		
30	70,686	589,048	18,750		
35	82,467	687,223	21,875	.1823	3/16
40	94 248	785,398	25,000	.2083	13/64
45	106,029	883,573	28,125	.2344	15/64
50	117.810	981,748	31,250	.2604	17/64
55	129,591	1,079.923	34,375	.2865	9/32
60	141,372	1,178,097	37.500	.3125	5/16
65	153,153	1,276,272	40,625	.3385	21/64
70	164,934	1,374,447	43,750	.3646	23/64
75	176,715	1,472,622	46,875	.3906	25/64
80	188,496	1,570,796	50,000	.4166	13/32
85	200,277	1,668,971	53,125	.4427	7/16
90	212,058	1,767,146	56,250	.4688	15/32
95	223,839	1,865,321	59.375	.4948	31/64
100	235,619	1,963,496	62,500	.5208	1/2
105	247,400	2,061,670	65,625	.5469	35/64
110	259,181	2,159,845	68,750	.5729	37/64
115	270,962	2,258,020	71,875	.5989	19/32
120	282,743	2,356,194	75,000	.6250	5/8

21-FT. DIAMETER CYLINDER.

Circumference, 65.9735; area, 346.3606.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure, Pounds.	Thickness, Dec. In.	Thickness, Frac. In.
10	25,977	216,475	6,563		
15	38,966	324,713	9,844		
20	51,954	432,951	13,126		
25	64,943	541,188	16,406		
30	77,932	649,426	19,688		
35	90,920	757,664	22,969	.1914	3/16
40	103,908	865,902	26,250	.2187	7/32
45	116,897	974.139	29,531	.2461	15/64
50	129,885	1,082,377	32,812	.2734	17/64
55	142,874	1,190,615	36,094	.3008	19/64
60	155,862	1,298,852	39,375	.3281	21/64
65	168,151	1,407,090	42,656	.3554	23/64
70	181,839	1,515,328	45,938	.3828	3/8
75	194,828	1,623,565	49,219	.4101	13/32
80	207,816	1,731.803	52.500	.4375	7/16
85	220,805	1,840,040	55,781	.4648	15/32
90	233,793	1,948,278	59,062	.4922	1/2
95	246,782	2,056,516	62,344	.5195	33/64
100	259,770	2,164,754	65,625	.5469	35/64
105	272,759	2,272,991	68,906	.5742	37/64
110	285,747	2,381,229	72,188	.6015	39/64
115	298,736	2,489,467	75,469	.6289	5/8
120	311,724	2,597,704	78,750	.6562	21/32

22-FT. DIAMETER CYLINDER.

Circumference, 69.1150; area, 380.1327.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure, Pounds.	Thickness, Dec. In.	Thickness, Frac. In.
10	28,510	237,583	6,875		
15	42,765	356,374	10,312		
20	57,020	475,166	13,750		
25	71,275	593,957	17,187		
30	85,530	712,749	20,625		
35	99,785	831,540	24,063	.2005	3/16
40	114,040	950,332	27,500	.2292	7/32
45	128,295	1,069,123	30,937	.2578	1/4
50	142,550	1,187,915	34,375	.2865	9/32
55	156,805	1,306,706	37,812	.3151	5/16
60	171,060	1,425,498	41,250	.3438	11/32
65	185,315	1,544,289	44,687	.3724	3/8
70	199,570	1,663,081	48,125	.4010	13/32
75	213,825	1,781,872	51,562	.4297	7/16
80	228,080	1,900,663	55,000	.4583	15/32
85	242,335	2,019,455	58,437	.4869	1/2
90	256,590	2,138,246	61,875	.5156	33/64
95	270,845	2,257,038	65,312	.5443	35/64
100	285,100	2,375,830	68,750	.5729	9/16
105	299,354	2,494,621	72,187	.6015	39/64
110	313,609	2,613,412	75,625	.6402	41/64
115	327,864	2,732,204	79,062	.6588	21/32
120	342,119	2,850,996	82,500	.6875	11/16

23-FT. DIAMETER CYLINDER.

Circumference, 72.2566; area, 415.4756.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure, Pounds.	Thickness, Dec. In.	Thickness, Frac. In.
10	31,161	259,672	7,187		
15	46,741	389,508	10,781		
20	62,321	519,344	14,375		
25	77,902	649,181	17,968		
30	93,482	779,016	21,562		
35	109,062	908,853	25,156	.2096	3/16
40	124,643	1,038,689	28,750	.2396	15/64
45	140,223	1,168,525	32,343	.2695	9/32
50	155,803	1,298,361	35,937	.2995	5/16
55	171,384	1,428,197	39,531	.3294	11/32
60	186,964	1,558,033	43,125	.3594	23/64
65	202,544	1,687,869	46,719	.3893	25/64
70	218,125	1,817,706	50,312	.4193	27/64
75	233,705	1,947,542	53,906	.4492	29/64
80	249,285	2,077,376	57,500	.4792	31/64
85	264,866	2,207,214	61,093	.5091	1/2
90	280,446	2,337,050	64,687	.5390	17/32
95	296,026	2,466,886	68,281	.5690	9/16
100	311,607	2,596,722	71,875	.5989	19/32
105	327,187	2,726,558	75,468	.6289	5/8
110	342,767	2,856,395	79,062	.6589	21/32
115	358,348	2,986,231	82,656	.6888	11/16
120	373,928	3,116,067	86,250	.7187	23/32

24-FT. DIAMETER CYLINDER.

Circumference, 75.3982; area, 452.3893.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure, Pounds.	Thickness, Dec. In.	Thickness, Frac. In.
10	33,929	282,743	7,500		
15	50,894	424,115	11,250		
20	67,858	565,486	15,000		
25	84,823	706,858	18,750		
30	111.788	848,230	22,500	.1875	3/16
35	118,752	989,602	26,250	.2187	7/32
40	135,717	1,130,973	30,000	.2500	1/4
45	152,682	1,272,345	33,750	.2812	9/32
50	169,646	1,413,716	37,500	.3125	5/16
55	186,610	1,555,088	41,250	.3437	11/32
60	203,575	1,696,460	45,000	.3750	3/8
65	220,540	1,837,831	48,750	.4063	13/32
70	237,504	1,979,203	52,500	.4375	7/16
75	254,469	2,120,575	56,250	.4687	15/32
80	271,434	2,261,947	60,000	.5000	1/2
85	288,398	2,403,318	63,750	.5313	17/32
90	305,363	2,544,690	67,500	.5625	9/16
95	322,327	2,686,061	71,250	.5938	19/32
100	339,292	2,827,433	75,000	.6250	5/8
105	356,256	2,968,805	78.750	.6563	21/32
110	373,221	3,110,176	82,500	.6875	11/16
115	390,186	3,251,548	86,250	.7187	23/32
120	407,150	3,392,920	90,000	.7500	3/4

25-FT. DIAMETER CYLINDER.

Circumference, 78.5398; area, 490.8739.

Height.	Capacity, Gallons.	Weight, Pounds.	Pressure, Pounds.	Thickness, Dec. In.	Thickness, Frac. In.
10	36,815	306,796	7,812		
15	55,223	460,194	11,719		
20	73,631	613,592	15,625		
25	92,039	766,990	19,531		
30	110,447	920,389	23,437	.1871	3/16
35	128,854	1,073,787	27,344	.2279	15/64
40	147,262	1,227,185	31,250	.2604	17/64
45	165,670	1,380,583	35,156	.2929	19/64
50	184,077	1,533,981	39,062	3255	21/64
55	202,485	1,687,379	42,969	.3581	23/64
60	220,893	1,840,777	46,875	.3906	25/64
65	239,301	1,994,175	50,781	.4232	27/64
70	257,709	2,147,573	54,687	.4558	29/64
75	276,117	2,300,971	58,594	.4883	31/64
80	294,524	2,454,370	62,500	.5208	17/32
85	312,932	2,607,768	66,406	.5534	9/16
90	331,340	2,761,166	70,312	.5859	19/32
95	349,748	2,914,563	74,219	.6185	5/8
100	368,155	3,067,962	78,125	.6510	21/32
105	386,563	3,221,360	82,031	.6836	11/16
110	404,971	3,374,758	85,937	.7161	23/32
115	423,379	3,528,156	89,844	.7487	3/4
120	441,786	3,681,554	93,750	.7812	25/32

CHAPTER V.

MECHANICAL PRINCIPLES.

IN the previous chapter it has been shown that the application of force as tension, compression, or shear, produces strain among the particles of which the body consists, and that this external pressure is resisted by the cohesive force of its fibres; also that the internal resistance of the particles depends upon their number and their arrangement in the cross-section. When weight or pressure is applied to such body as a beam or girder, two opposing forces are set in motion; one tending to cause rupture or the breaking of the beam through its cross-section, and the other exerting an opposing force of the fibre resistance depending in effect upon arrangement and tenacity. The tendency of the load applied to the beam is to produce "flexure" or bending, straining the fibres on the under side of the beam or producing tension among them, and compressing correspondingly the upper or outside fibres, both directly as their distance from the outer sides toward the centre of the beam. The strain which taxes to the maximum those most remote fibres from the central line, both by tension and compression, is gradually neutralized as the strain of tension and compression approach each other, and at the line of the cross-section where these two opposing forces meet, the fibres are at rest as regards each other, or are said to be in equilibrium, and at that line the fibres are neither under tension nor compression.

The line through the cross-section of any beam where the fibres are not strained is termed the "neutral axis" of the beam. In the case of all vertical loads, this neutral axis exists and passes through the centre of gravity of the beam cross-section parallel to the top and bottom faces of the beam.

Bending and Resisting Moments.—The effect of any vertical load, acting through the centre of gravity of the beam to produce flexure, is the amount of the load sustained and the point of application, or its leverage, as well the "bending moment" M at any cross-section of a beam, or the algebraic sum of the vertical forces on the left or right of the section, where the tendency of the forces is to cause motion by rotation around that point. The maximum bending moment occurs, of course, where the beam is most greatly strained. Without demonstration, the bending moment of a beam, $M = \frac{1}{8}Wl$; where $W =$ the total load and l its leverage.

The resistance offered by the fibres and their arrangement to the effects of the applied load is determined by the "resisting moment," R, of the beam, and is found by obtaining the algebraic sum of all the moments of the horizontal stresses producing tension and compression of the fibres, acting in opposite directions but parallel to each other. These moments are determined, with respect to the neutral axis, by adding together or summing up algebraically all the moments of all the unit stresses acting upon all the elementary areas of which the cross-section consists.

When this value equals that of the applied weight when multiplied by its leverage of action, called the "moment of rupture," or M, we have the equation, $R = M$, indicating equilibrium between the forces tending to cause rupture and those which offer resistance to the former forces.

Moment of Inertia.—In the consideration and design of beams, the effect of the shape or cross-section of the beam has to be taken into account and is analyzed by the aid of a

quantity termed the "moment of inertia," I, which, referred to the neutral axis of the beam, is the product of the square of the distance from that axis to all the elementary areas of the cross-section, and its value is determined by summing up the product of the elementary areas, multiplied by the square of their distances from the neutral axis, or solving Σaz^2 where Σ represents the summation, a the elementary area, and z its distance from the neutral axis.

Without demonstration, the resisting moment, R, of a beam is determined by dividing the moment of inertia, I, by the distance, as c, from the neutral axis to the extreme fibres; therefore the formula, $R = \frac{I}{c}$.

Modulus of Elasticity.—As has been said, not only does the cross-section of the beam, representing the arrangement of the fibres, have to be taken into consideration in determining the resistance offered by a given form to an external force, but the tenacity of those fibres or their cohesive force, and this last consideration deals with the relative ability to resist "elastic deformation" to the point of "ultimate elongation" and rupture. Provided none of the stresses exceed the "elastic limit" of the material, the elongation and deflection of beams can be computed.

The letter E is generally taken to represent the "modulus of elasticity" or the "coefficient of elasticity," representative terms expressing the ratio of "unit stress" to "unit deformation," and to be found by dividing the unit stress, as S, representing say, the stress in pounds per square inch, by the unit of elongation which, by experiment, has been found to follow the application of stress on different materials, as s; hence, $E = \frac{S}{s}$.

Under tension, and compression, experiment has determined that the coefficient or modulus E is practically the

same, while for shear stress, it is generally assumed at one-third less. It is further generally assumed that the stress under tension and compression when the elastic limit is reached is about six-tenths of the ultimate tenacity.

According to William Kent, A. M. M. E., one of the most recognized authorities on mechanical questions, the following are the

MODULI OF ELASTICITY FOR IRON AND STEEL.

Cast iron........	12,000,000 to 27,000,000 (?)
Wrought iron....	22,000,000 to 29,000,000
Steel..........	26,000,000 to 32,000,000.

Quoting from "Kent's Pocket Book": "The maximum figures given by many writers for iron and steel, viz., 40,000,000 and 42,000,000, are undoubtedly erroneous. . . . The modulus of elasticity of steel (within the elastic limit) is remarkably constant, notwithstanding great variations in chemical analysis, temper, etc. It rarely is found below 28,000,000 or above 31,000,000. It is generally taken at 30,000,000 in engineering calculations."

The values given above are generally approximated as follows:

Cast iron......	15,000,000	pounds	per	square	inch
Wrought iron..	25,000,000	"	"	"	"
Steel	30,000,000	"	"	"	"

When under tension or compression steel will stretch or shorten

$$\frac{1}{30,000,000}$$

part of its normal length for every pound per sectional inch in change of load.

The tendency of columns or struts under load is to fail by both compression and flexure, or bending, the column yield-

ing to the applied load, and deflecting laterally; the longer the column the greater the tendency to this lateral deflection or bending, and the greater the stresses upon the fibres of the concave side. The combined stress is very complex and difficult of demonstration, but it is pretty well established that the stress produced by such deflection increases directly as the *square* of the *length* of the beam.

In the discussion of columns, a quantity called the "radius of gyration" of the cross-section is an important factor in calculations, and, in the determination of the strength of a column or strut, represents the effect of the form of the column which is expressed by the square of the radius of gyration, or the moment of inertia of the section divided by its area, or $\frac{I - r^2}{A}$.

Radius of Gyration.*—Concerning this quantity, Trautwine says: "Suppose a body free to revolve around an axis which passes through it in any direction; or to oscillate like a pendulum hung from a point of suspension. Then suppose, in either case, a certain given amount of force to be applied to the body, at a certain given distance from the axis, or from the point of suspension, so as to impart to the body an angular velocity; or, in other words, to cause it to describe a number of *degrees* per second. Now, there will be a certain point in the body, such that if the entire weight of the body were there concentrated, then the same force as before, applied at the same distance from the axis, or from the point of suspension as before, would impart to the body the same angular motion as before. This point is the centre of gyration; and its distance from the axis, or from the point of suspension, is the *radius of gyration* of the body. In the case of *areas*, as of cross-sections of pillars or beams, the surface is supposed to revolve about an imaginary axis; and, unless

* Merriman defines the radius of gyration as "that quantity whose square is equal to the moment of inertia of the cross-section divided by its area," or $r^2 = I/A$ is the expression by which r^2 is to be computed. The student should observe that r has no connection with gyration, as I has no connection with inertia, in the case of sections of beams and columns. Radius of gyration is merely a technical name, which has unfortunately come into use, to denote the square root of the quantity I/A.

otherwise stated, this axis is the neutral axis of the area, which passes through its centre of gravity. The

"Radius of gyration = $\sqrt{\text{moment of inertia} \div \text{area}}$;

"Square of radius of gyration = moment of inertia $\div$ area.

"In a circle the radius of gyration remains the same, no matter in what direction the neutral axis may be drawn. In other figures its length is different for the different neutral axes about which the figure may be supposed to be capable of revolving. In rules for pillars the *least* radius of gyration must be used."

In the various handbooks periodically issued by the manufacturers of structural shapes, the radii of gyration and other elements of the usual sections are given, so that it is seldom necessary to compute the value of any of these from the formulæ.

The Gordon Formula for Strength of Columns.—Notwithstanding steel made into columns has shown a working value of 20 per cent. in excess of iron of the same shape, the formula for iron columns, invented by Lewis Gordon in 1840, after tests made before the British Board of Trade, continues in use, and is as follows:

ULTIMATE STRENGTH OF COLUMNS.

$$\text{Square bearing} = \frac{40000}{1 + \dfrac{(12l)^2}{36000r^2}}.$$

For safe resistance; quiescent loads, as for a building, divide by 4.

For safe resistance; moving loads, as in bridges, divide by 5.

In the above formula, the constant, 12, is to reduce the length 1, in feet, to inches; r, represents the *least* radius of gyration.

From Gordon's formula, the working value of the metal per square inch of section for columns of varying length is found; this, multiplied by the area of the section, gives the ultimate load.

To apply the Gordon formula, the length and section of the column must be known or assumed, and from the area of the cross-section the element "r" can be found by dividing the moment of inertia of the shape by its area, as has been shown; but in general "r" can be more conveniently found from any of the standard handbooks. In order to further lessen such computations, the following original table is given.

STRENGTH OF STEEL COLUMNS—BASED ON GORDON'S FORMULA.

Factor of safety of 4 used in table. 20 per cent. greater value assumed for steel than for iron columns.

l = length of column in feet.
r = least radius of gyration.
S = safe value of material per square inch of metal section.

$\frac{l}{r}$	S	$\frac{l}{r}$	S	$\frac{l}{r}$	S
2.0	11810	5.0	10908	8.0	9554
2.2	11774	5.2	10826	8.2	9456
2.4	11730	5.4	10746	8.4	9356
2.6	11683	5.6	10662	8.6	9260
2.8	11635	5.8	10578	8.8	9162
3.0	11586	6.0	10490	9.0	9062
3.2	11528	6.2	10400	9.2	8964
3.4	11468	6.4	10310	9.4	8864
3.6	11408	6.6	10218	9.6	8768
3.8	11346	6.8	10124	9.8	8670
4.0	11276	7.0	10032	10.0	8570
4.2	11208	7.2	9938	10.2	8474
4.4	11136	7.4	9842	10.4	8376
4.6	11060	7.6	9746	10.6	8280
4.8	10986	7.8	9650	10.8	8180

It is based upon the Gordon formula for iron columns, with a a higher value of 20 per cent, which from experiment has been definitely determined as applicable to steel members of

columns. A factor of safety of 4 has been used as being applicable to such structures.

To use the table, divide the length of the column in feet by the *least* radius of gyration, and from the corresponding ratio of the table find the unit strength of the material in pounds, which, multiplied by the combined area of the shapes, will give the safe load in pounds for a column of the required length and cross-section.

CHAPTER VI.

RIVETING.

IN structural metal-work, the usual method of uniting "plates" or of connecting "shapes" is by riveting.

The riveted joint is technically termed a "lap-joint" when one plate overlaps the other. It is a "butt-joint" when the two plates are brought together, their edges in contact, and the plates fastened by the use of a cover-strip or "welt," which overlaps both plates; when two such cover-strips are used, the one on the outside and the other on the inside of the two plates in contact, the joint is termed a "double-welt butt-joint."

Such joints are further distinguished as being "single-riveted" when a single row of rivets is used as fasteners for the two plates. It is a "double-riveted joint" when two rows of rivets are used; so, also, "triple-riveted" and "quadruple-riveted" when three and four rows respectively are used as fasteners; thus, a "triple-riveted, double-welt butt-joint" is one where three rows of rivets are used in making a joint between two plates, covered inside and out with covering-strips or "welts."

In the correspondence columns of the *Engineering News*, Mr. Freeman C. Coffin, M. Am. Soc. C. E., in discussing "Specifications for Stand-pipes," and referring to the character of joint, suggests some points where there is room for improvement. He writes as follows: "One is the method of

joining the plates. The present method of lapping both horizontal and vertical seams is awkward and unmechanical, and belongs more to the methods of the village blacksmith than those of precise and scientific mechanism. They should rather be like the accompanying sketch, taken from a paper read before the New England Water-works Association in 1893.

"In this sketch the horizontal seams are lapped, and the vertical seams made with butt-straps. This is a perfectly pre-

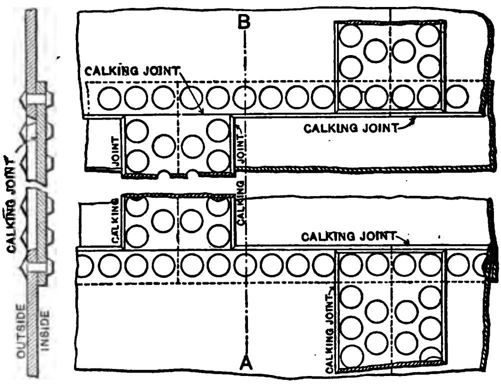


FIG. 4.—METHOD OF JOINING PLATES IN STEEL.

cise method, and requires no beating down or drawing out of the plates, and, in my opinion, would really cost no more than the old way. I use it now on plates over ½ in. in thickness, but should prefer to use it on all thicknesses."

Notwithstanding Mr. Coffin's opinion as to the relative cost, builders of stand-pipes will make quite a difference in the cost of a particular structure if the butt-joint is required, as it seems perfectly proper that they should do, for the reason that a butt-joint requires twice as many rivets as a lap-joint, because in the lap the rivet passes through both the plates, whereas in the butt-joint it passes through only one, so that there is necessarily an additional cost for punching or drilling, rivets, and driving.

There is no question, however, as to the increased value of a joint made as suggested by Mr. Coffin over the usual method, and it would seem as though the best practice should govern where the whole strength of the structure may depend upon its method of being assembled.

Efficiency of Riveted Joints.—The "efficiency" of a riveted joint is described as being the ratio of the strength of the *joint* to that of the *solid plate*. Thus, a joint is said to have a 70-per-cent. efficiency when the loss of strength, as compared with its ultimate strength, is 30 per cent.

In order to determine the efficiency of a riveted joint, it is necessary to know or to assume the following conditions:

(1) The tensile strength of the plate. (2) The diameter of the rivets used. (3) The unit resistance of these rivets, and their "pitch" or spacing, taken from centre to centre.

When proper values have been determined for the foregoing conditions, it has been found by practical tests and demonstrations that the efficiency of the several joints is approximately as follows:

Single-riveted joint..........	56 per cent. eff.
Double- " "	69 " " "
Triple- " "	75 " " "
Double-welt butt-joint........	87 " " "
Quadruple-riveted butt-joint..	95 " " "

One of the most interesting and practical discussions of the theory and practice of riveting with which the author is familiar, is contained in an address delivered to the students of Cornell College by Mr. J. M. Allen, president of the Hartford Steam Boiler and Insurance Co., and from which is quoted the following:

Single-riveted Joints (Fig. 5).—"In calculating the strength of a single-riveted joint we must know, *first*, what the tensile strength of the iron or steel plate is, from tensile

test; *second*, the diameter and pitch of the rivets; and *third*, the resistance to shearing per square inch of the material of which the rivets are made. On this latter requirement there has been no little discussion. It was formerly assumed, when only iron plates and iron rivets were used, that the shearing-resistance of a square inch of rivet was equal to the tensile strength of a square inch of the rivet itself or of the plate. That is, if we have iron of a tensile strength of 45,000 lbs. per square inch, the shearing-resistance of a square inch of rivet would be 45,000 lbs. On this assumption it would be only necessary to so arrange the diameter and pitch of rivets that

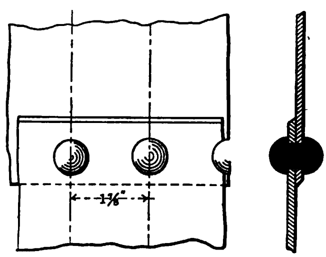


FIG. 5.—SINGLE-RIVETED JOINT.

the area of the rivet or rivets to be sheared should exactly equal the net section of plate to secure a perfect joint. Later experiments, together with the improvements in the manufacture of iron, and the introduction of steel, have changed these conditions relatively. While the shearing-resistance of the rivets per square inch has been, and even to-day is, by many assumed to be 45,000 lbs. per square inch, the assumption has arisen, no doubt, from the fact that rivets rarely shear. I have examined many exploded boilers, and the fractures have almost invariably been through the solid plate or along the line of rivets. It is very rare that the rivets shear. This, no doubt, arises from the fact that the pitch of the rivets was out of proportion to the net section of the plate. The old rule seemed to be: the more rivets, the stronger joint. There was, no doubt, a desire on the part of the boiler-makers to

make a tight joint, and they thought that if they pitched the rivets wider it would be difficult to caulk the joint so that it would be steam- and water-tight.

One would quite naturally assume that steel plates should be riveted with steel rivets, but such is not the usual practice. Most of the boilers now constructed in this country are made of steel plates, and they are largely riveted with iron rivets. In this country there have been comparatively few experiments on the strength of riveted joints made of steel plates and steel rivets, and as the general practice is to use iron rivets with both iron and steel plates, I confine myself here to the discussion of the iron rivet. I will say, however, that in England very careful experiments have been made, and a large percentage of strength is given to steel rivets over iron rivets. When the true value of the steel rivet is fully decided, and its use becomes general in this country, that value can be easily substituted for the value of iron rivets in the calculations of the strength of riveted joints, the other elements of the problem remaining the same.

What value, then, shall we give to the iron rivets when used in connection with steel or iron plates? In settling this question, I have not only been aided by the experiments of English engineers, but I have availed myself of experiments made on the large Emery testing-machine at the U. S. Arsenal at Watertown, Mass. These experiments have been made with American iron and steel, and hence will be valuable to us all in our practical work in this country. In a series of five experiments with steel plates and iron rivets, holes punched, the shearing-resistance per square inch was as follows: 39,740 lbs., 38,190 lbs., 36,770 lbs., 38,638 lbs., and 41,100 lbs. In view of these results, and other similar experiments, I assume 38,000 lbs. per square inch as the safe estimate of the single shearing-resistance of iron rivets in steel plates. Later experiments may change these figures

slightly. In these experiments the steel plate was 55,000 lbs. tensile strength per sq. in.

Assuming 38,000 lbs. as the safe estimate, we must decide upon the thickness of plate, diameter of rivet-hole, and pitch of rivets. In deciding upon these elements in the problem, we must so adjust the size and pitch of rivets as to make the shearing-resistance of the rivets as near the strength of net section as possible. I will assume the elements of the problem to be as follows:

Steel plate, tensile strength per square inch of section, 55,000 lbs.

Thickness of plate $\frac{5}{16}$ in. = decimal 0.3125.

Diameter of rivet-hole $\frac{13}{16}$ in. = decimal 0.8125.

Area of rivet-hole = decimal 0.5185.

Pitch of rivets $1\frac{7}{8}$ ins. = decimal 1.875.

Shearing-resistance of iron rivets per square inch = 38,000 lbs.

Then $1.875 \times 0.3125 \times 55{,}000 = 32{,}226$ lbs. = strength of solid plate.

$(1.875 - 0.8125) \times 0.3125 \times 55{,}000 = 18{,}262$ = strength net section of plate.

$0.5185 \times 38{,}000 = 19{,}703$ lbs. = strength one rivet in single shear.

Net section of plate is the weakest, therefore $18{,}262 \div 32{,}226 = 56.6$ per cent. efficiency of joint.

Double-riveted Joints (Fig. 6).—In double-riveted joints we find an accession of strength over single-riveted joints of nearly 20 per cent. This arises from the wider lap and the better distribution of the material. The rivets are pitched wider, and there is more rivet-area to be sheared, together with a larger percentage of net section of plate to be broken.

Steel plate, tensile strength per square inch of section, 55,000 lbs.

Thickness of plate $\frac{3}{8}$ in. = decimal 0.375.

Diameter rivet-hole $\frac{15}{16}$ in. = decimal 0.9375.

Area of rivet-hole = decimal 0,69.

Pitch of rivets $3\frac{1}{16}$ ins. = decimal 3.0625.

Shearing-resistance of iron rivets per square inch, 38,000 lbs.

Then $3.0625 \times 0.375 \times 55,000 = 63,164$ = strength of solid plate.

$(3.0625 - 0.9375) \times 0.375 \times 55,000 = 43,828$ lbs. = strength of net section.

$0.69 \times 2 \times 38,000 = 52,440$ lbs. = strength of two rivets in single shear.

Net section of plate is the weakest, therefore $43,828 \div 63,164 = 69.3$ per cent. efficiency of joint.

70 per cent. is usually assumed in practice.

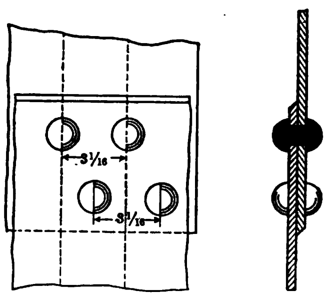


FIG. 6.—DOUBLE-RIVETED JOINT.

Triple-riveted Joint (Fig. 7).—In a triple lap-riveted joint we still gain in strength for reasons similar to those above.

Steel plate, tensile strength per square inch of section, 55,000 lbs.

Thickness of plate $\frac{3}{8}$ in. = decimal 0.375.

Diameter of rivet-holes $\frac{13}{16}$ in. = decimal 0.8125.

Area of rivet-hole = decimal 0.5185.

Pitch of rivets $3\frac{1}{4}$ ins. = decimal, 3.25.

Shearing-resistance of iron rivets per square inch, 38,000 lbs.

Then 3.25 × 0.375 × 55,000 = 57,031 lbs. = strength of solid plate.

(3.25 − 0.8125) × 0.375 × 55,000 = 50,273 lbs. = strength of net section plate.

0.5185 × 3 × 38,000 = 59,109 lbs. = strength of 3 rivets in single shear.

Net section of plate is weakest, therefore 50,273 ÷ 67,031 = 75 per cent. efficiency of joint.

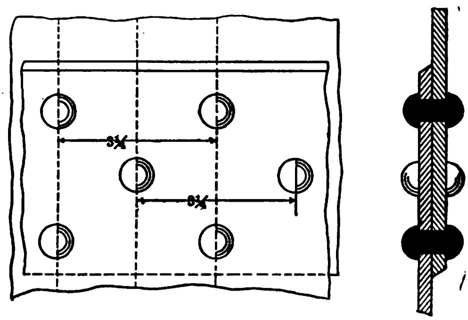


FIG. 7.—TRIPLE-RIVETED JOINT.

Double-welt Butt-joint (Fig. 9).—We now come to the double-welt butt-joint, triple-riveted.

I have selected this joint because we use it in practice where boilers of large diameters and high pressures are required.

In the double-welt joint a new element comes into the problem, viz., that of rivets in double-shear. Its inner welt is broader than the outer welt, and extends far enough beyond the former to enable us to introduce a third row of rivets, which are in single-shear, but also are in double-pitch. This increases the net section of plate, and also adds another rivet to be sheared. All the other rivets are in double-shear. The question now arises, What is the value of a rivet in double-shear? We have assumed, therefore, that the value of a rivet in single-shear was 38,000 lbs. per square inch.

Now, can we assume that the same rivet in double-shear has twice the value that it had in single-shear? It has been

assumed by some writers that such is the case, and up to this time most engineers allow a double value to rivets in double-shear. In the former the rivet is sustained by the plates above and below, while in single-shear the resistance is confined to one point.

An examination of the sheared sections of rivets in single-shear usually discloses a slight elongation in the direction of the force applied. The experiments on rivets in single-shear, and from which we get our data, have almost always been made on single-riveted joints, with narrow strips of iron, as shown in Fig. 8.

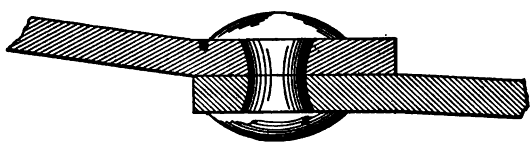


FIG. 8.

And it is reasonable to assume that there is a slight tendency in the rivet to lean in the direction of the force applied, which would account for the slight elongation of the sheared section in that direction. An examination of the sheared sections of rivets in double-shear shows little or no elongation. The rivets being supported by the plates above and below, the shear is direct, and the section is normal in form. Experiments made by the English Admiralty with $\frac{3}{4}$-inch rivets showed that the double-shear was about 90 per cent. stronger than the same diameter of rivet in single-shear. Chief Engineer Shock, U.S.N., found by experiment that the resistance of bolts of iron to single-shear was 40,700 lbs. per square inch, and in double-shear 75,300 lbs. This gives an increase of strength of 85 per cent. The results of numerous experiments, both in this country and in Europe, show the resistance to double-shear to be from 85 to 90 per cent. greater than the same rivets in single-shear. From the foregoing I assume 85 per cent. as a fair and safe estimate of

the excess of strength of rivets in double-shear over those in single-shear. We have already assumed that the resistance of rivets per square inch to single-shear is 38,000 lbs. If we add to this 85 per cent., we shall have 70,300 lbs. as the safe estimate of the resistance of iron rivets per square inch to double-shear. Further experiments may change these figures slightly, but I regard them as safe for use in all places where joints riveted with iron rivets are used. The use of the double-welt butt-joint in the construction of boilers is becoming quite common. This arises from the use of boilers of much larger diameter than those formerly used, and also the necessity for higher pressures on account of the introduction of compound engines.

With larger diameter and higher pressures, we find ourselves confronted with a very important problem. We must keep within the bounds of safety, for these large vessels are very destructive to life and property if we disregard the importance of good material, good workmanship, and the well-established factors of safety. It is not always safe to assume the highest results obtained by experimental tests. There will always be those who will insist upon higher pressures than safe rules will allow. Hence it becomes important that the consulting engineer shall thoroughly understand the principles of safe construction, and not allow himself to be moved in his judgment where the question of safety is involved. We will now apply the above data to the following problem:

Steel plate, tensile strength per sq. in. of section, 55,000 lbs.
Thickness of plate $\frac{3}{8}$ in. = decimal 0.375.
Diameter of rivet-holes $\frac{13}{16}$ in. = decimal 0.8125.
Area of rivet-hole = decimal 0.5185.
Pitch of rivets in inner rows $3\frac{1}{4}$ ins. = decimal 3.25.
Pitch of rivets in outer rows $6\frac{1}{2}$ ins. = decimal 6.50.
Resistance of rivets in single-shear = 38,000 lbs.
Resistance of rivets in double-shear = 70,300 lbs.

$6.5 \times 0.375 \times 55{,}000 = 134{,}062$ lbs. = strength of solid plate.

$(6.5 - 0.8125) \times 0.375 \times 55{,}000 = 117{,}304$ lbs. = strength of net section of plate at AB.

$0.5185 \times 4 \times 70{,}300 = 145{,}802$ lbs. = strength of 4 rivets in double-shear.

$0.5185 \times 38{,}000 = 19{,}703$ lbs. = strength of 1 rivet in single-shear.

This last result must be added to the strength of four rivets in double shear—thus, $145{,}802 + 19{,}703 = 165{,}505$ = shearing-strength of all the rivets. The net section of plate

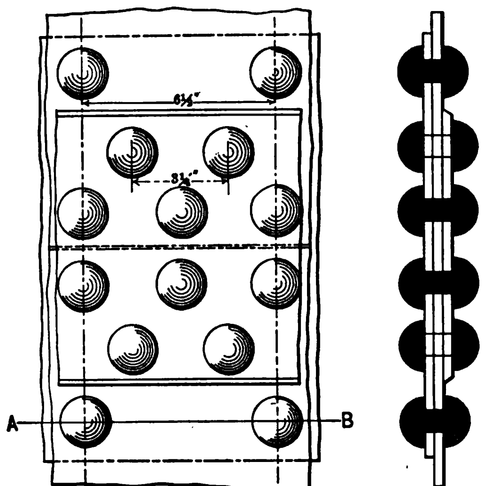


FIG. 9.—DOUBLE-WELT BUTT-JOINT.

is weakest; therefore, $117{,}304 \div 134{,}062 = 87.5$ per cent. efficiency of joint.

It will no doubt be observed that the strength of rivets in this joint is largely in excess of the strength of net section of plate, and the question will arise, Why increase the width of the inner covering-strip and add two more rivets? As stated above, this was done to increase the net section of plate at AB,

and thus increase the efficiency of the joint. If the inner welt or covering-strip had been of the same width as the outer one, the net section of the plate would have been greatly reduced, and the difference of strength between net section of plates and rivets would have been greater, thus reducing the efficiency of joint. The problem would be as follows:

$6.5 \times 0.375 \times 55{,}000 = 134{,}062$ = strength of solid plate.

$(6.5 - 0.8125 \times 2) \times 0.375 \times 55{,}000 = 100{,}546$ = strength of net section of plate.

$0.5185 \times 4 \times 70{,}300 = 145{,}802$ = strength of 4 rivets in double shear. Net section of plate is the weakest; therefore, $100{,}546 \div 134{,}062$ = only 75 per cent. efficiency of joint.

Again, it may be suggested: Why not dispense with one row of rivets in double shear, and extend the inner welt or covering-strip so that the outer row of rivets in double pitch and single shear could be used, thus increasing net section of plate as in the original problem, but reducing at the same time the shearing-resistance of the rivets?

The solution of this problem would be as follows:

$6.5 \times 0.375 \times 55.000 = 134{,}062$ = strength of solid plate.

$(6.5 - 0.8125) \times 0.475 \times 55{,}000 = 117{,}304$ = strength of net section.

$0.5185 \times 2 \times 70{,}300 = 72{,}901$ = strength of 2 rivets in double shear.

$0.5185 \times 38{,}000 = 19{,}703$ = strength of 1 rivet in single shear.

This last result must be added to the result of 2 rivets in double shear. $72{,}901 + 11{,}703 = 92{,}604$ = strength of all the rivets.

The total strength of the rivets is the weakest; therefore, $92{,}604 \div 134{,}062$ = 69 per cent. efficiency of joint.

It may be further suggested that a rivet of smaller diameter could be used. I will say that I have also considered such

a problem, but have come to the conclusion that the joint, as illustrated and described, for efficiency and freedom from leaks, is best. I will say here that a joint of this description was carefully made and tested on the Emery machine at the United States Arsenal at Watertown, Mass. The result of the test was two-twentieths of 1 per cent. of the calculation made, and the line of fracture was through the net section of plate at the outer row of rivets, as we had predicted."

Since the lecture delivered by Mr. Allen, in 1891, there has been rapid progress both in the manufacture and use of steel for structural purposes, and the practice of uniting steel plates with steel rivets has become the rule rather than the exception, although it seems that the great majority of metal-workers continue to be very conservative in assuming higher shearing-values for steel rivets, and while the steel rivet is used, calculations are made upon its efficiency without assuming much higher values than it has been the practice to give to iron rivets subject to shear.

In 1896 the United States Government made a series of tests upon riveted joints at the Watertown Arsenal. These experiments were made on joints formed of steel plate, and both iron and steel rivets.

An investigation of the reports shows the average shearing-value of steel rivets to have run as high as 55,000 lbs. per square inch for rivets of $\frac{3}{4}$-in. and $\frac{7}{8}$-in. diameters, and about 45,000 lbs. for steel bolts under the same conditions.

From these tests it would seem that the shearing-value of rivets in single-shear was about the same as the ultimate strength of steel rods under tension; and it would therefore seem that a higher working value for rivets might be established, and that for rivets in single-shear an ultimate value of 45,000 to 50,000 lbs. per square inch of metal would not be radical or likely to prove unsafe.

As has been shown, if the plate and rivet be given the same values, it would only be necessary to so arrange the diameter and pitch of rivet that the area of the rivets should equal that of the net section of plate to secure a perfect joint, but the ultimate value of plate steel is about 60,000 lbs., and that of rivet metal 50,000 lbs. per sq. in., and practice has further increased the difference between the metals by allowing only about 40,000 lbs. ultimate strength to rivet-rods under shear.

The area of the rivet-hole represents the true section of the rivet when driven, and therefore the area of the rivet-hole, multiplied by the shearing-value of the metal, gives the strength of the rivet.

The pitch of the rivet, representing a section of plate, multiplied by its thickness and the tensile strength of the metal, gives the strength of the solid plate, while the pitch of the rivet, or length of section, less one-*half* the diameter of the rivet-hole at *each* end of the section, or for both ends, the diameter of the rivet-hole, multiplied by the thickness of the plate and its ultimate tensile strength, will give the strength of the *net* section of plate. The relation of these values expressing the "efficiency" of the joint in per cent. is therefore found by dividing the greater value by the least.

Pitch of Rivets.—The pitch of the rivet is found by the formula

$$P = \frac{A \times S}{T \times Q} + D, \text{ where}$$

$P =$ Pitch of rivet,

$A =$ Area of rivet-hole in decimal of an inch,

$S =$ Shearing-value of rivet,

$T =$ Thickness of plate,

$Q =$ Tensile strength of plate,

$D =$ Diameter of rivet-hole in inches.

Where rivet is in more than single pitch, multiply by number of rivets in row.

Example.—Find the proper pitch for double-riveted joint, $\frac{1}{4}$-in. plate and $\frac{5}{8}$-in. rivet:

$$P = \frac{3712 \times 2 \times 40{,}000}{.2500 \times 60{,}000} + .6875 = 2.6671 \text{ or } 2\tfrac{5}{8} \text{ in.}$$

In the example above, 40,000 lbs. is taken as being a conservative value for a rivet in single-shear, and as allowing some latitude for irregularity in shop-work.

Size of Rivets in Relation to Thickness of Plates.—The determination of the size of rivet to be used as a fastener for certain thicknesses of plates is not governed by any hard and fast rule, but varies considerably in the practice of different manufacturers.

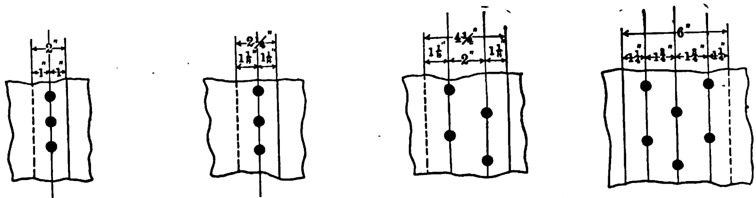
From investigation made by the United States Government, the relation of thickness of plates to diameter and length of rivets has been established by the Bureau of Construction and Repair, Navy Department, as follows:

Thickness of Plate, Inches.	Diam. of Rivet.		Corresponding Rivet-hole Area.			Length, Inches.
	In.	Dec.	In.	Dec.		
Less than $\frac{3}{16}$	$\frac{3}{8}$	.3750	$\frac{7}{16}$	.4375	.1503	$\frac{7}{8}$
$\frac{3}{16}$ to $\frac{1}{4}$	$\frac{1}{2}$	.5000	$\frac{9}{16}$	.5625	.2485	1
$\frac{1}{4}$ " $\frac{5}{16}$	$\frac{5}{8}$	.6250	$\frac{11}{16}$	.6875	.3712	$1\frac{1}{4}$
$\frac{5}{16}$ " $\frac{1}{2}$	$\frac{3}{4}$	.7500	$\frac{13}{16}$	.8125	.5185	$1\frac{3}{4}$
$\frac{1}{2}$ " $\frac{3}{4}$	$\frac{7}{8}$	.8750	$\frac{15}{16}$	.9375	.6903	$2\frac{1}{4}$
$\frac{3}{4}$ " 1	1	1.0000	$1\frac{1}{16}$	1.6250	1.0031	$2\frac{7}{8}$

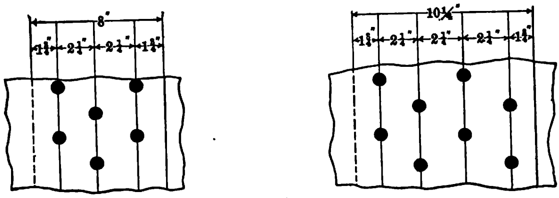
[NOTE.—Centres of rivets are spaced not less than $1\frac{3}{8}$ times their diameter from the edges. In double- and treble-riveting, their distance from centre to centre of rows (horizontal pitch) to be not less than $2\frac{1}{4}$ diameters in laps, and $2\frac{1}{2}$ diameters for straps.]

In the above table the length includes length of shank necessary to form the field-head measured under manufacturers' head, and for a "grip" equal to twice the thickness of plate assumed.

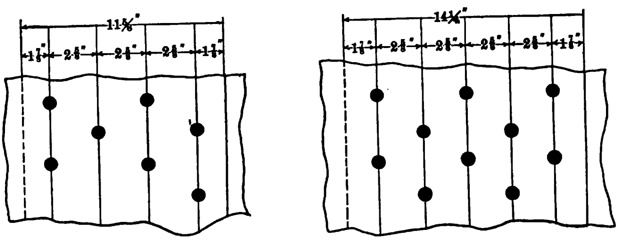
In order to facilitate calculations for water-tight metallic joints, the following table, providing an efficiency of joint suitable for metallic reservoirs, and an auxiliary diagram of details, has been designed by the author.



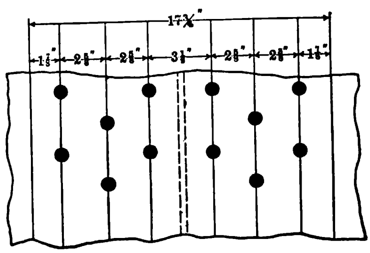
1/8" RIVET. DIMENSIONS OF LAPS USING 5/8" RIVETS.



LAPS USING 3/4" RIVETS.



LAPS USING 7/8" RIVETS.



BUTT STRAP—7/8" RIVETS.

FIG. 10.

RIVET CONNECTIONS—WATER-TIGHT METALLIC JOINT.

Thickness of Plate.	Weight per Foot.	Diameter of Rivet.	Length of Rivet.	Weight per 100.	Lap—No. Rivets in Vertical Row. 1	2	3	4	5	6	Vertical Pitch of Rivets. 1	2	3	4	5	6
3/16"	7.66	1/2"	1 1/4"	18.3	2"	......	......	......	......	......	1 1/2"					
1/4	10.20	5/8	1 1/2	34.6	2 1/4	4 1/4"	6"	......	......	......	1 3/4	2 5/8"				
5/16	12.76	5/8	1 3/4	36.7	......	......	6	......	......	......	......	......	3 1/4"			
3/8	15.30	5/8	1 3/4	36.7	......	......	6	......	......	......	......	......	2 3/4			
7/16	17.86	3/4	2.0	53.0	......	......	8	......	......	......	......	......	2 7/8			
1/2	20.40	3/4	2 1/8	54.4	......	......	8	......	......	......	......	......	3			
9/16	22.96	3/4	2 1/4	56.1	......	......	......	10 1/4"	......	......	......	......	......	3 1/4"		
5/8	25.50	3/4	2 3/8	57.7	......	......	......	10 1/4	......	......	......	......	......	3		
11/16	28.06	7/8	2 5/8	89.2	......	......	......	11 5/8	......	......	......	......	......	3 5/8		
3/4	30.60	7/8	2 3/4	91.3	......	......	......	11 5/8	......	......	......	......	......	3 1/4		
13/16	33.15	7/8	2 7/8	93.4	......	......	......	......	14 1/4"	......	......	......	......	......	3 3/4"	
7/8	35.70	7/8	3 1/4	99.8	......	......	......	......	......	......	......	......	......	......	......	3 3/4"
15/16	38.25	7/8	3 1/2	104.1	......	......	......	......	......	......	......	......	......	......	......	3 1/2
1	40.80	7/8	3 3/4	108.3	......	......	......	......	......	......	......	......	......	......	......	3 3/8

RIVET CONNECTIONS—WATER-TIGHT METALLIC JOINT.—*Continued.*

Thickness of Plate.	Weight per Foot.	Diameter of Rivet.	Length of Rivet.	Weight per 100.	Horizontal Pitch of Rivets.						Strap.		Per Cent. Efficiency of Joint.					
					1	2	3	4	5	6	Width.	Thickness.	1	2	3	4	5	
$\frac{3}{16}''$	7.66	$\frac{1}{2}''$	$1\frac{1}{4}''$	18.3									58.9					
$\frac{1}{4}$	10.20	$\frac{5}{8}$	$1\frac{1}{2}$	34.6		2″	$1\frac{3}{4}''$						56.7	73.8				
$\frac{5}{16}$	12.76	$\frac{5}{8}$	$1\frac{3}{4}$	36.7			$1\frac{3}{4}$								73.1			
$\frac{3}{8}$	15.30	$\frac{5}{8}$	$1\frac{3}{4}$	36.7			$1\frac{3}{4}$								71.9			
$\frac{7}{16}$	17.86	$\frac{3}{4}$	2.0	53.0			$2\frac{1}{4}$								71.7			
$\frac{1}{2}$	20.40	$\frac{3}{4}$	$2\frac{1}{8}$	54.4			$2\frac{1}{4}$								69.1			
$\frac{9}{16}$	22.96	$\frac{3}{4}$	$2\frac{1}{4}$	56.1				$2\frac{1}{4}''$								74.9		
$\frac{5}{8}$	25.50	$\frac{3}{4}$	$2\frac{3}{8}$	57.7				$2\frac{1}{4}$								72.9		
$\frac{11}{16}$	28.06	$\frac{7}{8}$	$2\frac{5}{8}$	89.2				$2\frac{5}{8}$								73.9		
$\frac{3}{4}$	30.60	$\frac{7}{8}$	$2\frac{3}{4}$	91.3				$2\frac{5}{8}$								71.2		
$\frac{13}{16}$	33.15	$\frac{7}{8}$	$2\frac{7}{8}$	93.4					$2\frac{5}{8}''$								75.0	
$\frac{7}{8}$	35.70	$\frac{7}{8}$	$3\frac{1}{4}$	99.8						$2\frac{5}{8}''$	$17\frac{1}{4}''$	$\frac{1}{2}''$						72.6
$\frac{15}{16}$	38.25	$\frac{7}{8}$	$3\frac{1}{2}$	104.1						$2\frac{5}{8}$	$17\frac{1}{4}$	$\frac{1}{2}$						73.2
1	40.80	$\frac{7}{8}$	$3\frac{3}{4}$	108.3						$2\frac{5}{8}$	$17\frac{1}{4}$	$\frac{9}{16}$						71.2

REMARKS.—Ultimate tensile strength of plate taken at 60,000 lbs. Rivets—single shear, 40,000 lbs.; double shear, 37,000 lbs. Horizontal joints single riveted—pitch of rivets, 4 × diameter of rivet. Lap—same as vertical pitch. Plates to $\frac{3}{8}''$ punched. Plates from $\frac{5}{8}''$ to $\frac{7}{8}''$ punched $\frac{1}{16}''$ less, and reamed. Plates over $\frac{7}{8}''$ drilled from solid. Length of rivets includes allowance for hand-driven head. Weight of rivets includes standard round head. Lap allows for bevel shear and trimming.

The sizes and spacing of rivets for marine-, boiler-, and tank-work, requiring water- and steam-tight joints, is somewhat different from that demanded for structural work, such as bridges, buildings, and towers. For structures of the latter type, the following general rules are applicable:

RIVET-SIZES AND SPACING FOR STRUCTURAL WORK. (DU BOIS.)

Diameter of rivet-hole: Not less than thickness of thickest plate through which it passes. For cross-girders, stringers, compression-members: $\frac{3}{4}$- to $\frac{7}{8}$-in. rivets.

General rule: Diameter of hole $= 1\frac{1}{4}$ thickness $+ \frac{3}{16}$ in.

Number of rivets: Divide total stress transmitted by joint by product of diameter of rivet by thickness of plate by safe bearing-value per square inch of rivet material.

For number of rivets to resist shear: Divide total stress by product of area of rivet, by safe shearing-value. (Shearing-values used in practice are 6000 to 7000 lbs. per square inch.)

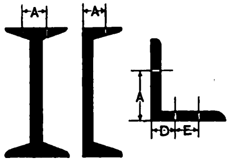
RIVET-SPACING FOR STRUCTURAL WORK.

Assume shearing strength equal to tensile strength.

$p =$ pitch; $d =$ diameter of rivet; $t =$ thickness of plate, and $a =$ section of rivet. $p = \frac{a}{t} + d.$

Practical restrictions: Rivets should not be closer than 3 diameters, nor more than 6 inches, centre to centre. In compression, never more than 16 times thickness of thinnest outside plate. Distance from centre of rivet-hole to edge, end, or next row of rivets should not be less than 2 diameters of rivet. The following table is the Carnegie Steel Company's practice for structural work:

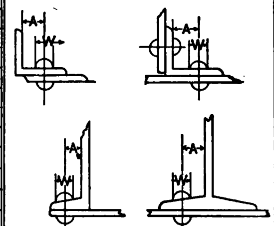
CARNEGIE STANDARD. SPACING AND DIMENSIONS OF RIVETS FOR FLANGES OF I^s-$[^s$ & L^s



METHOD OF INCREASING SECTION AREA.

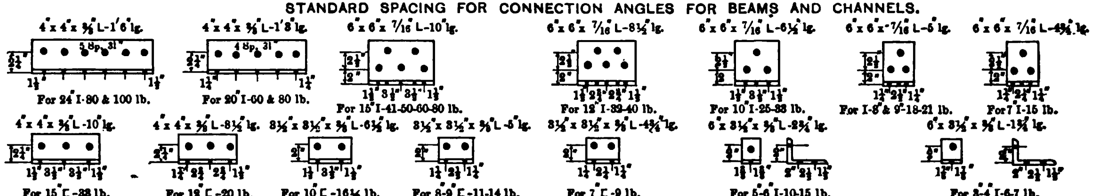
BEAMS				CHANNELS				ANGLES			PHŒNIX	
DEPTH	WEIGHT PER FOOT	A	MAX. DIA. OF RIVET	DEPTH	WEIGHT PER FOOT	A	MAX. DIA. OF RIVET	DEPTH OF LEG	MAX. DIA. OF RIVET	A	D	E
24"	80	4	¾"	15	33.	1⅞	¾	7"	1	3½	2½	2½
20"	80	4	¾	13	31.5	2.	¾	6"	1	3½	2¼	2¼
20"	64	3½	¾	12	20.	1⅝	¾	5"	1	2¾	2	1¾
15"	80	3½	¾	10	16.5	1½	¾	4½"	1	2½	1⅞	1½
15"	60	3¼	¾	9	14.	1⅜	¾	4"	1	2¼	1¾	1
15"	50	3	¾	8	11.	1¼	¾	3½"	1	2		
15"	41	3	¾	7	9.5	1⅛	⅝	3¼"	⅞	1¾		
12"	40	3	¾	6	8.	1.	⅝	3"	⅞	1¾		
12"	32	2¾	¾	5	6.5	1.	⅝	2¾"	¾	1½		
10"	33	2¾	¾	4	5.5	1.	½	2½"	¾	1⅜		
10"	25	2½	¾	3	5.	1.	⅜	2¼"	⅝	1¼		
9"	21	2½	¾					2"	⅝	1⅛		
8"	18	2¼	¾					1¾"	½	15/16		
7"	15	2¼	⅝					1½"	½	13/16		
6"	13	2	⅝					1⅜"	½	¾		
5"	10	1¾	½					1¼"	½	11/16		
4"	7	1½	½					1⅛"	½	⅝		
3"	6	1⅜	⅜					1"	⅜	9/16		

CLEARANCE FOR DRIVING L^s-$[^s$ & I^s



MINIMUM DISTANCE $A_1 = \frac{1}{4} + \frac{1}{2}$ OF WIDTH W

STANDARD SPACING FOR CONNECTION ANGLES FOR BEAMS AND CHANNELS.



CONNECTIONS FOR 3"-4"-5" & 6" I^s APPLY ALSO TO CHANNELS. ALL HOLES FOR ¾" BOLTS OR RIVETS.

FIG. 11.

CHAPTER VII.

DESIGNING.

HAVING formed a clear conception of the principles explained in the preceding chapters, it is possible to consider intelligently the subject of designing metallic reservoirs and their supporting substructures.

By the use of the various tables, applicable to included sizes, the study of suitable design is greatly facilitated and simplified. In the general scheme of a water-supply system, where storage and gravity supply is included, in the absence of a sufficiently elevated natural location, the necessity for some form of metallic reservoir to supply or supplement the deficiency is apparent.

From the general requirements as to pressure and storage, the dimensions of the structure will be determined.

From the analysis of "Stand-pipe Statistics," page 8, it has been found that the average domestic pressure, as required in the United States, is 61.2 lbs. per sq. inch. If this pressure is satisfactory to the designing engineer, as shown on page 65, the corresponding height or head is approximately 142 ft., which would be the required height of the stand-pipe. Under ordinary conditions, however, the local topographical condition is likely to afford certain convenient natural elevations, advantage of which may be taken to reduce the height of the metallic reservoir, which height, supplemented by the natural elevation, will give the required pressure.

In the case of a particular design, where there occurs an

available natural elevation of 22 or 23 ft., representing a pressure of say 10 lbs., the difference between this and the required pressure of 61.2 lbs. is 51.2, and which we see (page 65) represents a head of 120 ft. approximately; and we therefore determine to erect a stand-pipe 120 ft. in height, and, having assumed the height, the capacity required fixes the dimensions.

The question of capacity is settled most arbitrarily; but, in general, it is the usual practice to provide a storage or reserve supply which will permit the temporary stoppage of the pumping-engines for repairs, etc., for a given number of hours. In small towns, particularly where a lighting-plant may be operated in conjunction with the water-works, it is sometimes deemed desirable to provide sufficient storage to supply the ordinary consumption during the day by the pumping done at night, making only one set of firemen and engineers necessary for both plants. Another determining element in fixing the capacity of storage and the corresponding size of the reservoir is, of course, the item of cost and the amount of money available. As has been shown, the widest range of practice in the matter of diameter, height, and corresponding capacity exists; but, for the purpose of discussion and analysis, we will assume that a metallic reservoir of 400,000 U. S. gals. is required. The height having been taken as 120 ft., from the table (page 71), we see that, for the given height and capacity, the diameter will be approximately 24 ft., the actual capacity for the cylinder, 120 × 24 ft., being 407,150 U. S. gallons.

Strain-sheet.—In designing such a structure, through the employment of the principles previously enunciated, the details can be specified; their correctness demonstrated mathematically, or shown graphically.

Usually a graphic demonstration of the correct principles of construction is shown by a "strain-sheet," similar to that

shown below. . . . The line $H'B$ is first drawn, at right angles to which the vertical line HH is laid off. By any convenient scale, point off or divide the horizontal and vertical lines into equal subdivisions.

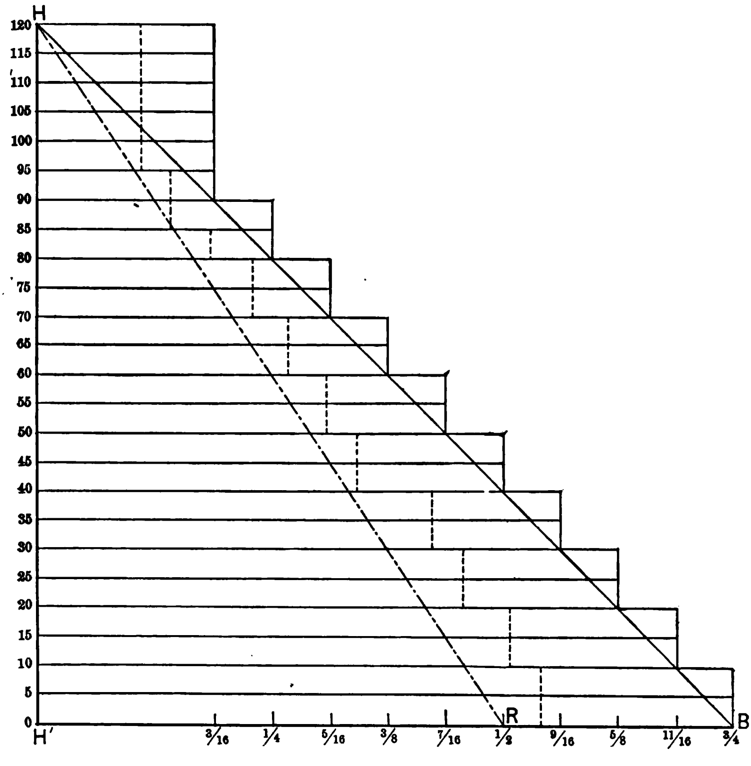


FIG. 12.—STRAIN-SHEET, 24 × 120-FT. STAND-PIPE.

The subdivisions of the horizontal line can be taken to represent the decimal or fractional parts of an inch, the latter being usually the case, as the thickness of steel or iron plate is generally considered in fractions of an inch. The value to be given the horizontal subdivision will depend upon the intentions of the designing engineer; that is, whether he

intends to construct his stand-pipe of plate advancing by 64ths, 32ds, 16ths, or 8ths of an inch. Usually the thickness of the plates to be used in the ascending sections or rings are decreased by 16ths; but, in close calculations, the scale is taken at 32ds, and in which case the value of any subdivision would be one thirty-second on the horizontal line.

The value given to equal subdivisions of the vertical line $H'H$ can be taken at decimals of 100 ft., and represent the height of each panel or ring taken in the clear—that is, between laps. The height of the rings is generally uniform, but is entirely arbitrary, the limiting height being determined by cost and convenience of handling; thus, a stand-pipe with a greater number of shorter rings would require a greater number of connecting joints, with increased cost of rivets, punching, and driving, as well as decreased efficiency in the general strength of the structure, than one with greater height of ring and fewer joints; but the larger the plate which is to be used in the construction of the ring, the more difficult it becomes to handle, both on account of the increased weight and the trouble given by the wind catching the broad expanse of plate metal, swinging and swaying it in the most troublesome manner as it is being hoisted into place.

It has been found from practice, both in shop- and field-work, that a 5-ft. segment is a very convenient height, and therefore the practice of making the rings 5 ft. in the clear seems to be in general use. Assuming that this height will be adopted, the value of the subdivisions of the vertical scale would be 5 ft.

The increasing height on the vertical scale, in multiples of five, is usually indicated as shown on the strain-sheet, as is also the increasing thickness on the horizontal scale, advancing by 16ths, 32ds, etc., as may be determined in advance.

Application of Mechanical Principles.—The formula for arriving at the theoretical thickness of plates is explained on

page 67, and calculations suited to a wide range of heights and diameters of metallic cylinders have been given, so that between these ranges it is only necessary to revert to the tables to find the required theoretical thickness of the metal in fractions and decimals of an inch corresponding to the required height and capacity.

Thickness of Plate.—Considering a 24-ft. × 120-ft. standpipe, the theoretical thickness of the lower plate is seen to be $\frac{3}{4}$ of an inch.

Determining to advance by 16ths, twelve subdivisions of the horizontal line equal $\frac{3}{4}$ of an inch thickness of plate. Draw the diagonal line $H'B$, which is a line which indicates the theoretical thickness of the plate from zero at H, and where the thickness and strength of a piece of letter-paper is capable of resisting the pressure of the water, to B, where $\frac{3}{4}$ of an inch of steel, having a tensile strength of 60,000 lbs., with a factor of safety of 4, and a rivet-efficiency of $\frac{2}{3}$ the ultimate strength of the plate, is required to safely resist the hydrostatic pressure of 51.97 lbs. per square inch.

From the subdivisions of the vertical line $H'H$, draw perpendicular lines parallel to the base-line, with a distance apart of 5 ft. by the assumed scale, and with each length equal to the theoretical thickness of the plate, measured by the scale of the base. The length of these lines, representing the theoretical thickness of the plate, can be determined mathematically by the formula given, or from the table, as was done when establishing the thickness of the lower plate; but, to simplify this process, the length of each horizontal line can be determined graphically by terminating that line at the intersection formed by vertical lines, projected from the scale of the base, but which are not usually indicated except to complete the parallelogram.

If the parallelogram as thus formed lies inside of the diagonal line, the plate of which it is intended to construct

the ring is less than the required theoretical thickness demanded by the formula for the assumed conditions. If the parallelogram projects beyond the diagonal, the plate has greater thickness and strength than is theoretically necessary to resist the hydrostatic pressure at that point, the projecting area representing the excess of thickness and weight of the plate metal, and to that extent increasing the cost of the structure; in the same way the area included in the section between the diagonal and the vertical line when the latter is within the diagonal represents the proportion of insecurity. Obviously, the nearer the vertical projected line, intersecting with the horizontal, approaches the diagonal, the more nearly are the theoretical conditions of thickness of plate to applied pressure complied with; hence, in graphic design, the decrease in thickness of plate, corresponding with reduced pressures, should be shown as rising like steps along the diagonal, the foot of each rise just touching the diagonal line, and the three intersecting lines forming triangles whose area represents the excess of strength and plate metal beyond the theoretical requirements. This will be clearly understood by a slight study of the strain-sheet on page 106.

Joint Efficiency.—It is also customary to indicate upon the strain-sheet, graphically, the joint efficiency, or the percentage of strength of the joint as compared with the strength of the plate, showing by vertical dotted lines in each section the ratio of strength which the specified character of the joint bears to the strength of the solid plate.

In the formula for determining the thickness of the plate to resist safely the applied pressures, it was assumed that $\frac{1}{3}$ of the strength of the plate would be lost by punching and riveting; hence the line indicating the relative efficiency of the joint, or the "rivet-efficiency line," should be drawn to represent 66.6 per cent. of the theoretical strength of the plate as indicated by its thickness as measured on the scale or base-line

H'B. Thus, where the scale of the base is taken in 16ths, $\frac{3}{4}$ of an inch thickness will be represented by twelve subdivisions, which, multiplied by 66.6 per cent., gives 7.99 as the distance of the point where the rivet-efficiency line cuts the base to the point *H'*.

Draw the dotted diagonal *H–R*.

For each ring or panel the distance of each vertical dotted line from the dotted diagonal will graphically demonstrate the excess or decreased strength of that particular joint more or less than 66.6 per cent. As in the explanation of the proper relation of plate thickness to the diagonal theoretical line of strength, so the dotted vertical, showing rivet efficiency of the particular vertical joint, should not fall very far on either side of the 66.6 per cent.-rivet-efficiency line in any section or ring; otherwise the joint will be too weak for safety in the one case or unnecessarily strong, entailing increased cost, in the other.

It has been previously explained how the efficiency of a riveted joint was determined, and from the formula deduced a set of tables has been calculated; it is therefore only necessary to inspect the strength and efficiency of any joint as shown in the table, and to adopt and specify the character of joint, giving the requisite percentage of strength; then for any ring or section whose thickness is known and indicated on the vertical scale, multiply the number of subdivisions representing that thickness by the per-cent. efficiency of the accepted joint, and the result can be used to plot the point where the vertical dotted line should be drawn, as was done to establish the point *R* on the base-line *H'B*.

The strain-sheet given for the 24-ft. × 120-ft. stand-pipe, and further above explained and described, is frequently more or less elaborated to include other details, and is sometimes so complete as to render further specifications unnecessary for designing. Further details for this stand-pipe are given on the following page:

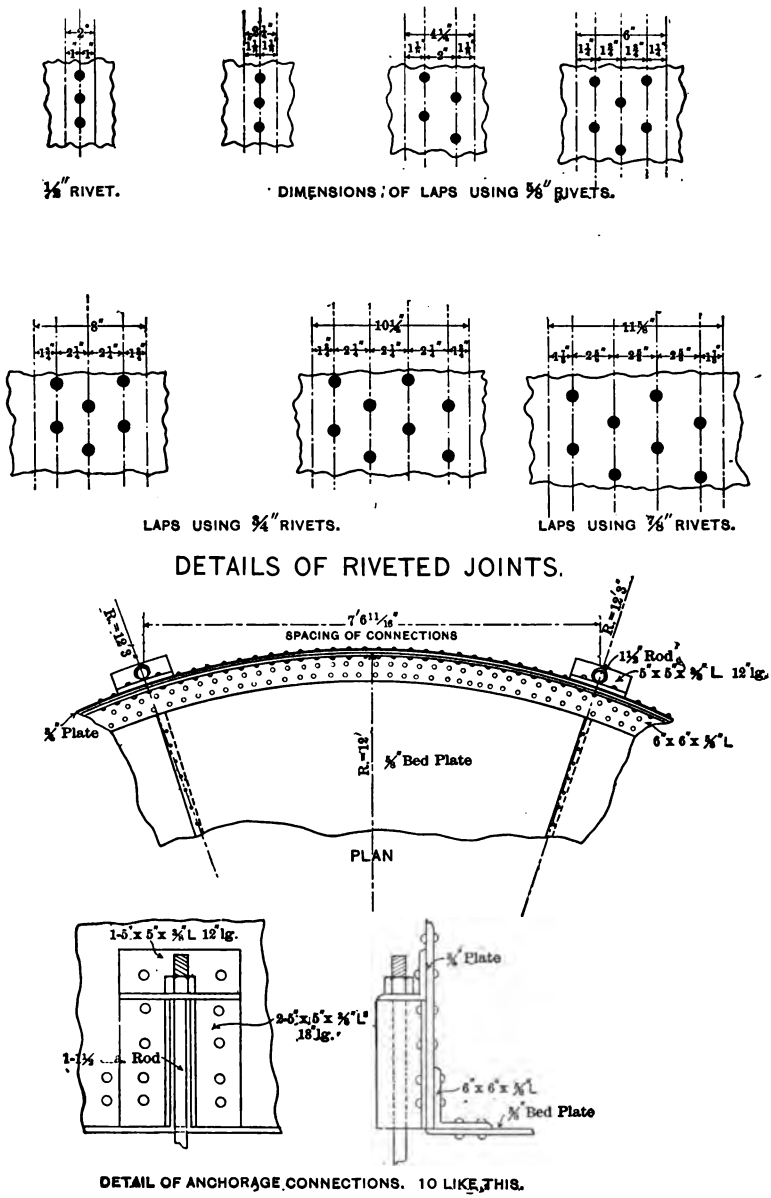

⅝″ RIVET.
DIMENSIONS OF LAPS USING ⅝″ RIVETS.
LAPS USING ¾″ RIVETS.
LAPS USING ⅞″ RIVETS.
DETAILS OF RIVETED JOINTS.
7′6 11/16″
SPACING OF CONNECTIONS
R.=12′3″
1⅛″ Rod
5″x 5″x ⅜″L 12″lg.
⅜″ Plate
6″x 6″x ⅜″L
⅝″ Bed Plate
R.=12′
PLAN
1-5″x 5″x ⅜″L 12″lg.
2-5″x 5″x ⅜″L 18″lg.
Rod
⅜″ Plate
6″x 6″x ⅜″L
⅝″ Bed Plate
DETAIL OF ANCHORAGE CONNECTIONS. 10 LIKE THIS.

FIG. 13.

Bed-plate and Connections.—In calculations for the thickness of the "bed-plate" or the plate which is to form the bottom of the cylindrical stand-pipe, the moment of the weight of the column of water, acting through the centre of gravity and applied at the centre of the circle, would be found by multiplying the weight by its leverage, the radius of the circle, and the thickness of the plate to resist this stress would be found as explained; but in stand-pipes the bed-plate rests upon and is supported by the subfoundation, so that it is only necessary to provide a plate which can be satisfactorily joined to the shell. In practice where the shell-plate, bottom ring, is $\frac{1}{2}$ in. or over in thickness, the thickness of the bed-plate is assumed at $\frac{3}{4}$ the thickness of the shell; where the bottom ring is less than $\frac{1}{2}$ in., the bed-plate is taken as the same thickness as the shell. In large stand-pipes the bed-plate sheets are cut economically to represent segments of the circle, are riveted together in the field, and joined to the shell by some form of "angle" or "L" curved to radius. The length of the legs of the angle are determined by the character of riveting required, sometimes it being sufficient to single-rivet both legs to the shell- and bed-plate respectively; sometimes the shell is double- and the bed-plate single-riveted; sometimes both are double-riveted, hence the comparative lengths of the angle-legs. The thickness of the angle is usually a a mean between the thickness of the shell- and bed-plates; thus, in the 24-ft. × 120-ft. stand-pipe the lower ring of the shell is $\frac{3}{4}$ in.; the bed-plate would be made $\frac{9}{16}$ in., and the thickness of the angle used for connection $\frac{5}{8}$ in.; as both the shell- and bed-plate are to be double-riveted, a 6 in. × 6 in. $\frac{5}{8}$-in. standard angle is required. The connecting-angle is sometimes placed inside and sometimes outside of the cylinder, but as the pressures are from the inside, the outside angle location is preferred, as the bed-plate then extends beyond the shell, and the angle riveted on acts as a brace, and the plate and leg

of the angle give that much additional stability to the structure. Some engineers prefer to flange the shell- and bed-plate, making a flanged joint instead of the angle-joint as described. Where it is unnecessary to extend the area of the base by the use of angles and web-plates, and the simple angles are used, as shown, the outer arrangement of the connecting-angle is impossible, and the connection is necessarily made on the inside.

Details.—As has been said, the hydrostatic pressure at the top of a tank being zero, the thickness and strength of a sheet of paper would be sufficient to control and restrain the pressures and water; but, in stand-pipes of any size, the thickness of the top rings is usually $\frac{1}{4}$ in., and never less than $\frac{3}{16}$ in. These thicknesses are used to provide for the weakening of the plates by oxidation or rusting of the metal, and also to resist the action of the wind, to successfully resist which it is usual to provide some "stiffener" at the top, usually an angle riveted to the inner or outer circumference of the cylinder, the horizontal leg being used to fasten and support an ornamental cresting, generally of malleable iron, cast in segments, and bolted to the angle.

In the records of stand-pipe failures, several large structures have suffered partial or total collapse during high winds, which were seen to roll the metal at the top into a cone-shape, similar to the twisting of a piece of paper into a taper. This action of the wind is not very well understood, and therefore the elements of the top angle are arbitrarily assumed in general, but it may be approximately correct to assume that the wind acts upon the exposed surface of a stand-pipe as upon a beam uniformly loaded, the total weight being

$W = d \times h \times 30 \div 2$, which weight we will assume is also uniformly applied at the top of the stand-pipe upon a beam represented by the diameter of the stand-pipe, then for the safe resisting-moment $\dfrac{R - Wl}{8}$.

Taking some standard angle whose elements are given, find its safe resistance for a span equal to the diameter of the stand-pipe, and to which add the additional resistance due to the thickness of the plate metal, and its length—the diameter—and width, taken as the length of the vertical leg of the angle; summing these two results will approximately determine the ratio of strength of the built shape to the pressure to be resisted. For this investigation of required angle for 24-ft. $\times$ 120-ft. stand-pipe, we find that a suitable stiffener will be an angle 7 ins. $\times$ $3\frac{1}{2}$ ins., weight 17 lbs. per lin. ft., where $\frac{3}{16}$ in. steel plate is used to make top ring.

This style of finish for the top of a stand-pipe, while in general use, is subject to criticism in that it is uncovered, and in some waters the sunlight quickly forms organic growths, while the angle without the cresting is an inviting roosting-place for birds—the writer having seen dozens of buzzards roosting upon the tops of stand-pipes so constructed; again, in cold climates an uncovered surface is objectionable on account of the greater tendency of the water to freezing, several recorded failures being ascribed in part to this cause. A better construction is to provide for a light plate-metal cover, supported upon radial rafters of light angle or channel shapes, the rafters being bent to project vertically below the top of the stand-pipe and forming stiffeners for that portion of the structure.

In addition to these stiffeners spaced at regular intervals, a light horizontal stiffener should be provided, set 12 or 18 inches below the top; and, if a Z shape is specified, a suitable support for a painter's trolley is thus secured, which will be found most convenient.

For purposes of inspection a ladder capable of safely sustaining a weight of not less than 1000 lbs. should be designed, and is sometimes used both inside and out—that for the outside terminating 10 ft. above the base of the structure, to prevent

mischievous or malicious persons from having too ready access to this facility. Such ladders may be composed of two side-bars, 2 ins. × $\frac{5}{16}$ in., with $\frac{3}{4}$-in. diameter rungs spaced 12 to 18 inches, which may also be a suitable spacing for the side-bars. Such ladders are generally built in sections at the shop, and are riveted to the sides of the stand-pipe at intervals of 10 to 12 feet with light angle-clips.

As it is sometimes necessary to empty the stand-pipe and to remove deposits, it is necessary to provide some kind of manhole near the base, which is usually of elliptical form, with plates, arches, and bolts, and of such dimensions as to provide easy ingress for a workman.

A suitable connection for the supply-pipe must also be arranged for, its dimensions being governed largely by the size of the inlet-pipe; the connection is usually a short bell-mouth section, flanged at both ends, the flange to be in contact with the plate to be curved to radius while the other end is planed for a standard flange-connection with the inlet-pipe, the first section of which generally has both a flange and bell end.

Methods of Anchorage.—Beside these connections, suitable connections for the anchor-rods must be designed and the number and size of rods determined.

The method of proportioning the anchor-rods was given at length, page 63, and as applied to the particular anchorage for 24-ft. × 120-ft, stand-pipe, using the principle of moments, we find, roughly, the weight of the empty stand-pipe to be 85 tons; the moment of this weight, or the resisting-moment, is 85 × 12 = 1020 foot-tons.

The overturning-moment of the wind is 24 × 120 = 2880 sq. ft. × 30 lbs. pressure, = 43.2 tons, into its leverage, 60 ft. = 2592 foot-tons; the tank is therefore unstable. Using iron rods of 40,000 lbs. ultimate fibre stress, reduced by a factor of safety of 4, we have 10,000 lbs. per sq. inch of rod-

area. Assuming $1\frac{7}{8}$ ins. as a suitable size, the area by the unit stress gives a product of 13.8 tons which each rod would exert in tending to keep the tank in position, and if 10 rods were used, the holding-down force would be 138 tons.

If 10, $1\frac{1}{2}$-in. steel rods of 60,000 T.S. were used, their holding-down value would be 133 tons with the same factor of safety.

A standard hexagon nut for a $1\frac{1}{2}$ in. bolt measures 3.18 ins. on its long diameter, so the rods could not be set closer than 1.59 ins. to the outer circumference of the cylinder, whose plate being $\frac{3}{4}$ in. thick, the radius from rod centre to centre of cylinder could not be less than 12 ft. $2\frac{11}{32}$ ins.; but as these nuts must be tightened with a wrench, we will give a little clearance by pitching them 12 ft. 3 ins., which would represent the lever-arm for determining the moment of the rods; hence, 133 tons $\times$ 12.3 ins. = 1629 foot-tons downward resisting-moment, which must be added to the same moment exerted by the weight of the metal, which has been found to be 1020 foot-tons; therefore the total downward moment of resistance is 2649 foot-tons, with an overturning-moment of the wind 2592 foot-tons; hence 10 $1\frac{1}{2}$-in. steel rods, pitched as explained, would have an excess strength of 57 foot-tons as represented by a comparison of the vertical and horizontal moments of the structure. This can be shown graphically.

TOWER AND TANK, WEST TAMPA, FLA.

CHAPTER VIII.

DESIGNING—CONTINUED.

In the general scheme of a water-supply plant, where storage is required and to be obtained only by the erection of a metallic reservoir, it is sometimes deemed expedient to secure a suitable elevation by constructing the tank upon a supporting tower. Such towers are made in many ways and of various materials, brick, wood, and metal being most generally used.

The choice of such substructure is determined by the conditions of capacity, cost, and local surroundings.

As to the question of capacity, the same considerations apply as those explained previously for stand-pipes.

The height of the tank superstructure may be considered as representing the minimum and maximum desirable or limiting pressures, hence it is argued that a stand-pipe has a large column of water which is useless except to support the *effective head* of the water above the minimum desirable pressure as determined in feet, and that the effective column may be more economically supported by an open substructure such as a steel tower. Arguments are also presented that the lower volume of water in a stand-pipe being useless except for purposes of support, it is objectionable from the fact that it is stagnant and the greater volume of water is more liable to become affected by organic growths. This argument is controverted upon the assumption that the temperature of

the water is constantly changing and therefore all sections of the column are equally fresh.

It is a fact, however, which is used to the best purposes by builders of this type of structure, that the record of failures show that few towers have failed as compared with the collapse of stand-pipes.

While this is true, it is also a fact that in the United States there are very many more stand-pipes in existence than towers and tanks, but on account of the comparatively small increased cost of securing a greater area of bearing-surface for the support of the structure, and also from the fact that by the wide spread of the supporting columns of a tower, the stability of the structure can be so increased that the resultant of the overturning-moment of the wind and the moment of the weights falls well within the figure limited by the spread of the columns, where the same resultant could only be secured for a stand-pipe by an abnormal area of base.

The local character of the bearing-soil exerts a considerable influence upon the selection of either type of structure, and this factor should be carefully considered in connection with the discussion of foundations as explained in the succeeding chapter.

A comparative investigation of these two types of structure will be given here briefly.

A 20-ft. × 112-ft.. stand-pipe has a capacity of 264,000 U. S. gals. A tank 20 ft. × 42 ft. with a conical or hemispherical bottom will contain, approximately, 100,000 gals.

If a reserve supply of 100,000 gals. be thought ample, and a 20-ft. × 42-ft. tank as described be erected upon a 70-ft. tower, the effective heads and corresponding pressures would be the same in both the stand-pipe and tower and tank for the first 42 feet from the top. If the maximum and minimum pressures be taken at 48.5 lbs. and 30.3 lbs., corresponding to 112 and 70 feet head, the lower 70 feet of the stand-pipe

contains 164,000 gals. of water which exerts a pressure less than the minimum decided upon.

Comparing the two structures upon a basis of cost, the stand-pipe contains approximately 50 tons of material, while the tower and tank will average approximately 35 tons of metal. If a satisfactory bearing can be secured by loading the soil with 2.5 tons per square foot, the stand-pipe will require about 130 cubic yds. of masonry, and the tower and tank (four supporting columns) will not average much more than 48 cubic yds. From these quantities a very rough basis of cost would be to assume the value of the stand-pipe at $5000 and the tower and tank at $4000, both including foundations.

If, after a careful consideration of the conditions both from an engineering and financial standpoint, it be determined that a tower and tank type of reservoir is preferable, the dimensions of the tank being assumed from reasoning analogous to that given in considering these factors in stand-pipe design, a strain-sheet is prepared as explained in the preceding chapter, but which will necessarily be modified, as will be explained hereafter, as far as the thickness of the lower ring and bottom plates are concerned; the conditions for their determination being changed.

In small railway water-supply tanks, flat or horizontal bottoms are usually provided, supported upon wooden sills or I beams of iron or steel, attached to the upper deck of the supporting structure. In such cases, the thickness of the lower ring is that determined by the formula, but the thickness of the bottom plate will depend upon the spacing of the beams or sills.

In cities or towns where the tower and tank is to be erected for public supply, some other form of bottom is generally specified, for the reason that other forms require somewhat less material: it is easier to secure and maintain water-tight joints; all parts of the bottom are accessible making sub-

sequent and necessary painting possible; the stresses are less than in the flat bottom; the conical, hemispherical, or compound shaped bottom is more symmetrical and pleasing to the eye, and last, the action of the effluent exerts an automatic scour or self-cleaning effect upon the bottom plates, preventing sedimentary deposits, which are sufficient—as has been shown in the discussion of flat-bottomed stand-pipes—to make it necessary to provide some form of manhead permitting ingress for removal of the deposit at intervals. For these reasons, the subsequent discussion of suitable bottoms will be limited to this type.

Theoretical Consideration of Thickness of Bottom Plates.—In order to determine the theoretical thickness of the bottom plate, the principles of the formula

$$\frac{d \times h \times 62.5}{2} \div \frac{60{,}000 \times 12 \times \frac{3}{8}}{4}$$

can be used with some modification, for the quantity h will not represent the height of the cylinder, as the bottom must be riveted to the lower ring of the shell *above* the base, and as the solidity of a cone is the area of the base into one-third its perpendicular height, the quantity h is the depth of the cylinder to the point where the bottom is riveted to the shell and one-third the height of the cone-shaped bottom. Where the bottom is of the hemispherical, conical, or compound type, it is obvious that with each succeeding ring, from the shell of the tank downward toward the outlet, the diameter of the circle becomes smaller with each successive joint, which reduced value represents the quantity d in determining the theoretical thickness of the corresponding metal rings. Thus, if a 42-ft. cylinder 20 ft. in diameter and with a cone-shaped bottom $7\frac{1}{2}$ ft. deep or high, and which is riveted to the shell 2 ft. above the base of the cylinder, is being considered, the quantity h is $42\frac{1}{2}$ ft. Again, the quantity "12," representing 12 inches of height along the cylinder, or a 12-in. ring, must

be changed, for in riveting the bottom to the shell, and where double riveting is used, the contact of ring would probably be only about 6 in., which should be substituted in the formula, hence we have for the 20 × 42-ft. cylinder,

$$\frac{20 \times 42.5 \times 62.5}{2} \div \frac{60,000 \times 6 \times \frac{3}{8}}{4},$$

which gives .442, or about $\frac{7}{16}$ inches as the theoretical thickness of the bottom plate at the point where it is to be riveted to the shell, and which is further decreased in the succeeding rings composing the bottom in accordance with the changing value *d*.

The Riveted Girder.—As such bottoms, being riveted to the tank-shell, depend upon the strength of bottom ring of the cylinder for support of the entire weight of water, the plate itself is incapable of distributing this weight to the supporting columns, having an insufficient bearing-area, therefore some form of girder must be designed, the properties of which will now be discussed.

With a "built" girder, as with any simple beam, its ability to support a load depends upon the strength and arrangement of its fibres, limited by its distance between supports. If the entire weight is to be supported upon a girder resting upon four columns at equidistant points along the circumference of the circle with which the girder corresponds, its length is the length of the circular arc subtending an angle of 90 degrees, or, in other words, its length is one-quarter of the circle; but in practice, the effective length of the girder may be taken as the distance between supports, or the "long chord" of the quadrant, which, for any included or central angle may be found from the formula $C = 2R \times \sin \frac{1}{2} \Delta$.

Where four supporting columns are used, $C = 2R \times .70711$.
For six supports .... $C = 2R \times .51504$.
For eight supports .... $C = 2R \times .38268$.

In the case under consideration, where $R = 10$ ft., the length of the long chord, taken to represent the effective length of the girder between supports, is 14.1422 ft.

With the 20×42-ft. tank under consideration, the weights transmitted to the girder are approximately as follows:

Weight of water in tank..................	428.7 tons.
Weight of material in tank...............	18 "
	446.7 tons.

Dividing total "dead load" by the length, taken at 62.83, the dead load per linear foot is approximately 7.1 tons.

In addition to this constant or dead load due to the weights, the pressure of the wind at times transmits stresses to portions of the structure as a variable or "live load."

From the principles of moments and the strength of materials, this pressure or maximum pressure may be found from the formula,

$$P = \frac{Ml}{2I},$$

which is a formula for determining the maximum pressure due to the wind applied over the area of the base, but as these pressures are transmitted to the girder, the unit stress due to the wind can be found by dividing this quotient by the length of the girder as measured along the circumference of the cylinder.

Using the quantities being considered in the formula, the variable load is found to be 70 tons, which, divided by the circumference 62.8, adds approximately 1.1 tons to the constant load of 7.1 tons, making the unit combined stresses 8.2 tons per lin. ft. of girder.

Also from the principle of moments and strength of materials, a formula for the safe load upon a beam supported at either end and uniformly loaded has been deduced,

$$W = 8\frac{SR}{L},$$

where W = required safe load;

" S = unit fibre stress of material;

" R = moment of resistance $\frac{(I)}{c}$;

" L = effective length between supports.

For angles and other ordinary and standard shapes, the elements I and c have been computed and may be found in almost any of the handbooks issued by manufacturers to their customers; but for such compound shapes as the riveted girder suitable for stand-pipe work, these elements must be computed unless those given in the following original table (p. 127) are found satisfactory. As the principles of moment are applicable to areas as well as to weights, this principle applied to areas of the elements of the compound shape can be used in an equation from which the value of c, or the distance from the neutral axis, passing through the centre of gravity of the shape, to the most remote fibres, can be determined.

As has been said, I and c, for the elementary angles may, in general, be obtained from any standard handbook.

For the rectangular web, c is one-half h, while I of the web is the same as for any rectangle, $\frac{bh^3}{12} = I$.

The summation of the products of the elementary shapes into the square of their distance from the neutral axis of the compound shape, gives the moment of inertia I for the riveted girder, while the moment of resistance R of the girder is, as has been explained, $R = \frac{I}{c}$.

Such girders having practically a quiescent load, it is considered good practice to allow flange-strains of 15,000 lbs. per square inch of net section, and 11,000 lbs. for the vertical

shearing-strains of the web, or for all practical purposes in this connection, 13,500 lbs. may be assumed as the allowable unit stress for this compound shape; hence the formula,

$$W = 8\frac{SR}{L} \text{ may be written.}$$

$$\text{Safe load in tons} = \frac{4.5 \times R}{L}.$$

This represents the safe load in tons distributed over the entire span, and must be divided by the span in feet to arrive at the safe unit stress in tons per linear foot of girder.

For the convenience of investigations of riveted girders, the following original tables have been calculated and inserted.

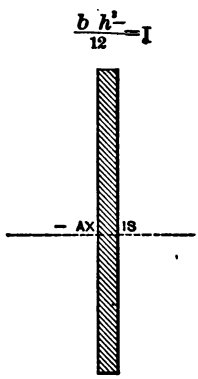


FIG. 14.

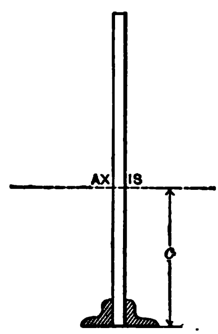


FIG. 15.

MOMENT OF INERTIA OF RECTANGLES.

Depth in Inches.	Width of Rectangle in Inches.						
	1/4	5/16	3/8	7/16	1/2	9/16	5/8
36	972	1215	1458	1701	1944	2187	2430
42	1543	1929	2315	2701	3087	3473	3859
48	2304	2880	3456	4032	4608	5184	5760
54	3280	4100	4920	5740	6561	7381	8201
60	4500	5625	6750	7875	9000	10125	11250

TABULATED ELEMENTS OF THE RIVETED GIRDER.

Moment of Inertia I, Depth of Neutral Axis c, and Modulus of Rupture R.

Depth In.	Flange In.	Angle Dimensions.	Th. Plate.	I	c	R
36	$6\frac{1}{4}$	$3 \times 3 \times \frac{1}{4}$	1/4	2268.9	22.0	103.1
42	"	"	"	3232.1	25.0	129.3
36	$7\frac{5}{16}$	$5 \times 3\frac{1}{2} \times \frac{5}{16}$	5/16	3521.5	22.8	154.4
42	"	"	"	4957.7	25.9	191.4
36	$8\frac{5}{16}$	$4 \times 4 \times \frac{5}{16}$	"	3482.7	22.8	152.8
42	"	"	"	4936.4	26.1	189.1
36	$7\frac{3}{8}$	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$	3/8	3735.2	22.4	166.8
42	"	"	"	5322.3	25.6	207.9
36	$9\frac{3}{8}$	$3\frac{1}{2} \times 4 \times \frac{3}{8}$	"	3934.8	22.7	173.3
42	"	"	"	5581.2	25.9	215.5
36	$10\frac{3}{8}$	$3\frac{1}{2} \times 5 \times \frac{3}{8}$	"	4267.1	23.0	185.5
42	"	"	"	6082.3	26.4	230.4
36	$12\frac{3}{8}$	$4 \times 6 \times \frac{3}{8}$	"	4853.5	23.5	206.5
42	"	"	"	6854.4	26.9	254.8
36	$7\frac{7}{16}$	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$	7/16	3873.3	21.9	176.8
42	"	"	"	5587.6	25.1	222.6
36	$8\frac{7}{16}$	$3 \times 4 \times \frac{5}{16}$	"	3396.5	21.4	158.7
42	"	"	"	4977.0	24.6	202.3
48	"	"	"	6952.0	27.7	250.9
36	$10\frac{7}{16}$	$3 \times 5 \times \frac{5}{16}$	"	3652.8	21.8	167.6
42	"	"	"	5319.4	25.0	212.7
48	"	"	"	7415.2	28.2	262.9
36	$12\frac{7}{16}$	$3\frac{1}{2} \times 6 \times \frac{3}{8}$	"	4714.5	22.9	205.9
42	"	"	"	6732.5	26.2	256 9
48	"	"	"	9230.5	29.5	312.9
36	"	$6 \times 6 \times \frac{7}{16}$	"	6723.9	23.9	281.3
42	"	"	"	9420.5	27.4	343.8
48	"	"	"	12637.1	30.8	410.3
36	$6\frac{1}{2}$	$4 \times 3 \times \frac{5}{16}$	1/2	3652.9	21.0	173.9
42	"	"	"	5359.6	24.1	222.4
48	"	"	"	7546.7	27.3	276.4
36	$8\frac{1}{2}$	$3 \times 4 \times \frac{5}{16}$	"	3589.5	21.1	170.1
42	"	"	"	5285.8	24.2	218.4
48	"	"	"	7440.4	27.3	272.5
36	$10\frac{1}{2}$	$3 \times 5 \times \frac{5}{16}$	"	3819.4	21.4	178.4
42	"	"	"	5638.7	24.7	228.3
48	"	"	"	7865.3	27.7	283.9
36	$12\frac{1}{2}$	$3\frac{1}{2} \times 6 \times \frac{3}{8}$	"	4816.2	22.4	215.0
42	"	"	"	6987.2	25.8	270.8
48	"	"	"	9583.2	28.9	331.6
36	$6\frac{9}{16}$	$4 \times 3 \times \frac{5}{16}$	9/16	3862.3	20.8	185 7
42	"	"	"	5706.8	23.9	238.7
48	"	"	"	8034.8	26.9	298.7
36	$8\frac{9}{16}$	$3 \times 4 \times \frac{5}{16}$	"	3799.6	20.9	181.8
42	"	"	"	5614.7	23.9	234.9
48	"	"	"	7951.6	27.0	294.5
36	$10\frac{9}{16}$	$3 \times 5 \times \frac{5}{16}$	"	4024.7	21.2	189.8
42	"	"	"	5937.1	24.3	244.3
48	"	"	"	8366.9	27.4	305.3
36	$12\frac{9}{16}$	$3\frac{1}{2} \times 6 \times \frac{7}{8}$	"	10020.6	24.9	402.4
42	"	"	"	14049.8	28.6	491.3
48	"	"	"	18556.3	31.9	581.7

In a compound riveted girder, it is usual to assume that the flange sustains the horizontal and the web all of the vertical strains due to the load; the flange acts under tension, and the web is subject to sheer stresses. The amount of sheer contributed to the web equals one-half the total weight. Besides the tendency to fail by shearing, there is a disposition of the web to fail by flexure or lateral bending, or buckling, to prevent which, vertical stiffening plates or shapes are riveted to the web on one or both sides at regular intervals; where the thickness of the web-plate is less than one-sixtieth of its depth, such auxiliary pieces become necessary. The spacing of these stiffeners is usually made about equal to the depth of the girder.

The compound shape made up of the web and auxiliary plates may be regarded as a vertical column or post under compression and subject to the principles of deflection of columns. The strength of the column may be calculated by means of the Gordon formula, which will give the unit value of the metal for columns of variable lengths. This unit stress into the area of the cross-section in square inches will give the ultimate or safe load in pounds. The compressive stresses due to the load and to be resisted by the strength of the column are exerted diagonally along the fibres of the web, which, between columns, acts in tension, the several parts of the compound shapes acting like members of a bridge-truss, the columns taking the compression stresses and the diagonals the tensile stresses of the structure.

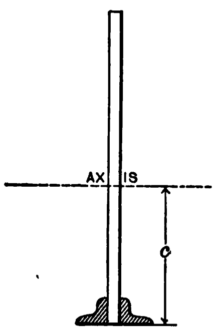


FIG. 16.

Beside the Gordon formula there exists numerous other formulæ for determining the strength of columns, largely

based, as is Gordon's, upon the results of Hodgkinson's experiments, or else modifications of Rankine's theoretical treatment of the subject. Merriman's development of the Rankine formula and to which he has given the name "Rational Formula for Columns," is of such frequent use that it is given place below, and his table applicable to steel columns with fixed ends inserted.

Merriman's Rational Farmula for Columns (*Eng. News*, July 19th, 1894):

$$C = \frac{B}{1 - \frac{nB}{\pi^2 E}\frac{l^2}{r^2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$B = \frac{C}{1 + \frac{nC}{\pi^2 E}\frac{l^2}{r^2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

B = unit-load on column = total load $P \div$ area of cross-section A; C = maximum compressive unit-stress on the concave side of the column; = length of the column; r = least radius of gyration of the cross-section; E = coefficient of elasticity of the material; $n = l$ for both ends round; $n = \frac{4}{9}$ for one end round and one fixed; $n = \frac{1}{4}$ for both ends fixed. This formula is for use with strains within the elastic limit only; it does not hold good when the strain C exceeds the elastic limit.

Prof. Merriman takes the mean value of E for timber = 1,500,000; for cast iron = 15,000,000; for wrought iron = 25,000,000, and for steel = 30,000,000, and $\pi^2 = 10$ as a close enough approximation. With these values he computes the following table from formula (1):

STEEL COLUMNS WITH FIXED ENDS.

Unit-load.	Maximum Compressive Unit-stress C.							
$\frac{P}{A}$ or B	$\frac{l}{r} = 20$	$\frac{l}{r} = 40$	$\frac{l}{r} = 60$	$\frac{l}{r} = 80$	$\frac{l}{r} = 100$	$\frac{l}{r} = 120$	$\frac{l}{r} = 140$	$\frac{l}{r} = 160$
7,000	7,020	7,070	7,150	7,270	7,430	7,650	7,900	8,230
8,000	8,020	8,090	8,200	8,380	8,570	8,770	9,200	9,650
9,000	9,030	9,110	9,250	9,450	9,730	10,090	10,550	11,140
10,000	10,030	10,130	10,310	10,560	10,910	11,360	11,810	12,710
11,000	11,040	11,160	11,380	11,690	12,110	12,670	13,410	14,370
12,000	12,050	12,200	12,450	12,820	13,330	14,020	14,930	16,130
13,000	13,060	13,230	13,530	13,970	14,580	15,400	16,500	17,990
14,000	14,070	14,250	14,610	15,130	15,850	16,830	18,150	19,960
15,000	15,080	15,310	15,710	16,310	17,140	18,290	19,870	22,060

The design of the cross-section of a column to carry a given load with maximum unit-stress C may be made by assuming dimensions, and then computing C by formula (1). If the agreement between the specified and computed values is not sufficiently close, new dimensions must be chosen, and the computation repeated. By the use of the table the work will be shortened.

The formula (1) may be put in another form which in some cases will abbreviate the numerical work. For B, substitute its value $P \div A$, and for Ar^2 write I, the least moment of inertia of the cross-section; then

$$I - \frac{P}{C} r^2 = \frac{nPl^2}{\pi^2 E} \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

in which I and r^2 are to be determined.

For example, let it be required to find the size of a square oak column with fixed ends when loaded with 24,000 lbs. per square inch. Here I = 24,000, C = 1000, $n = \frac{1}{4}$, $\pi^2 = 10$, E = 1,500,000, $l = 16 \times 16$, and (3) becomes

$$I - 24\, r^2 = 14.75.$$

Now let x be the side of the square; then

$$I = \frac{x^4}{12} \text{ and } r^2 = \frac{x^2}{12},$$

so that the equation reduces to $x^4 - 24x^2 = 177$, from which x^2 is found to be 29.92 square inches, and the side $x = 5.47$ inches. Thus the unit-load B is about 802 lbs. per square inch."

These principles and tables are of general application. In applying them to a suitable design for riveted girder of 20×42-ft. tank, whose total load is 516.7 tons, to be supported at four points, with an effective distance of 14.14 ft. between supports, the first element to be determined is the web, in which there should be a co-relation to the thickness of the bottom plate and the plate forming the ring just above the girder. As the web must sustain the weight of the water which is applied through the bottom plates, where practicable the web should approximate the thickness of the bottom plate, which has been found to be $\frac{7}{16}$ in., theoretically determined. For this investigation, select from the table some compound shape whose web is $\frac{7}{16}$ in. for a given height, which determines R, also found from the table; then

$$\text{Safe load in tons} = \frac{4.5 \times R}{L}.$$

For experiment, selecting $R = 410.3$, and the given length L between supports, 14.14 ft.,

Safe load in tons $= \dfrac{4.5 \times 410.3}{14.14} = 130.6$ tons; and since the actual constant and variable load of the tank, water, and wind has been found to be 129.2, the shape corresponding to $R = 410.3$ may be accepted as suitable.

Now, as the thickness of the web is less than $\frac{1}{60}$th of its

height, vertical stiffeners must be supplied along its length of 62.83 ft., which, reduced to inches and divided by 48, representing the spacing as determined by the height of the girder, gives as a quotient 16, and the actual spacing in inches from centre to centre of these 16 columns will be $47\frac{1}{8}$ inches.

The total load carried between girder-supports has been found to be 129.2 tons, and the shear-stress being equal to one-half the total load, or 64.6 tons, this must be sustained by the four columns which are to be spaced along the section of the girder between the main supports, so that each of these stiffener-columns must be designed to resist approximately 16 tons. Sometimes single angles are thus used as stiffeners, but more frequently, for purposes of utility and ornamentation, a light balcony is designed to surround the tank, in which case two light angles with a web-plate are selected, riveted back to back to the web; and the compound shape to the shell of the tank, which is the web of the compound riveted girder. At the top of the girder these angles are bent over to form a horizontal support for the floor of the balcony, and the web included between the angles is sheared to a triangular shape, the whole shape making a secure and ornamental brace for the platform, as well as serving to stiffen the web of the riveted girder.

Since in the investigations of columns, the *least* radius of gyration is used, in considering the radius of gyration of the compound shape, that due to the rectangular portion of the girder-web, represented by the thickness of the plate, being of small moment, need not be considered; but in obtaining the *area* of the compound stiffener, the area of this rectangle, or that portion of the girder-web covered by the angle or angles used as stiffeners into its thickness, must be added to the area of the angle or angles to obtain the area of the compound stiffener. In order to have as great a radius of gyration as possible in the direction of the applied stress, if the

angle is not regular, the narrower leg is riveted to the web, allowing the longer leg to project.

From a table giving the elements of standard angles, find the least radius of gyration in inches, by which divide the length of the column in feet. From the table Strength of Columns find the nearest approximate ratio of length in feet to radius in inches, in column $\frac{l}{r}$ which gives the safe unit-stress of the material, which multiplied by the area of the compound stiffener, will give the safe number of pounds which the column is capable of sustaining.

Applying these principles to secure a column capable of safely resisting the given weight, 16 tons, for experiment select a $2 \times 2 \times \frac{3}{16}$-in. angle; two such angles are intended to be riveted back to back to a $\frac{3}{16}$-in. web-plate. For the angle selected, the radius of gyration is found to be 0.62; the length of the web in feet being 4, then $\frac{l}{r} = \frac{4}{.62} = 6.4$, and from the table Strength of Steel Columns the corresponding ratio is found to be 10310 lbs. per square inch of metal section.

The area of the two angles is $.72 \times 2 = 1.44$; The area of the web included between them is $2 \times .1875 = .3750$, while the area of the section of the principal girder covered by the two angles and the small web is

$$\tfrac{7}{16} \text{ in.} = .4375 \times 4.1875 = 1.8320.$$

Summing these several areas,

Area 2 angles	1.44
" included web	.3750
" covered section of girder	1.8320
Total area compound section	3.6470

The unit-value of the metal being theoretically 10310, $A \times S$ = safe weight, $3.6470 \times 10310 = 37600$ lbs. or safe

weight in tons = 18.8; as 16 tons is the weight to be imposed upon each column, although slightly in excess of the actual weight, the elements of this compound section may be selected and used.

The stresses being greatest next the supports, the riveting of the girder-flange should be closer near those points, and an empirical rule is to space the rivets 3 ins. for a distance equal to the depth of the girder, or for 48 ins. in this case. The spacing of the rivets for intermediate distances may be made about equal to 16 times the thickness of the plate, or 6 inches. The size of such rivets are taken at from $\frac{3}{4}$ to $\frac{7}{8}$ in.; in the case under consideration, $\frac{7}{8}$-in. rivets should be used.

Supporting Columns.—Instead of supporting the load at four points, and designing a girder capable of safely carrying all of the imposed stresses, a reduction of the length of span and consequent decrease of the size and weight of the members of the riveted girder is sometimes considered, and is to be accomplished by increasing the number of supporting columns; or the length of the span may be reduced by designing short, diagonal struts, usually two for each column, thus reducing the length of span correspondingly for each strut thus supplied; two such struts to each column of a four-column tower would give twelve bearing-points in place of four, and the girder and its members could be correspondingly reduced, but this latter method of reducing the length of span is open to objection on account of the eccentricity of loading and the multiplication of members and joints.

With the usual four- or six-column tower the supporting columns are generally of the "built column" class, or columns formed by riveting together certain shapes. This class of work is performed at the shops, and the lengths so constructed are to be afterwards assembled in the "field," or place where the structure is to be erected, the only riveting necessary then being that at the "panel-points" or connec-

tions. Such columns are usually formed of channels or angles riveted to suitable plates, the channels being "laced" together to prevent individual weakness and to make all parts of the combined shape act as a unit. A favorite shape in general use at this time is the "Z"-bar column, or a set of four "Z"-bar shapes, riveted to a web-plate. This built column, in steel, is now manufactured by nearly all of the great steel works. This form of column possesses so many advantages for building purposes that it sprang into general use. In lengths ranging from 64 to 88 radii, from careful tests, an average ultimate resistance of 35,650 lbs. was determined. These results are more favorable than those for any other open column. Their great adaptability for making connections with other columns, struts or members, and their accessibility for inspection, painting or repair, together with their comparative cheapness and small number of rivets required to connect the individual bars together, make this a most excellent shape for such service as the supporting column of a tower and tank structure.

To provide a suitable design for columns and other members of a tower, the weights or stresses imparted to each must first be determined. Upon a skeleton diagram, drawn to any convenient scale, and indicating compression-members in heavy, and tensile members in light, lines, the stresses as found for each member should be indicated upon the diagram. To secure stability of position of the structure without spreading the columns over a very great area, and thus increasing the stresses upon the compression-members and the length of all members, some convenient inclination is given the columns, and the usual method of anchorage is employed to maintain the equilibrium.

An inclination or "batter" of one in ten is a very convenient inclination for computations, and is very generally used.

As the strength of any column has been shown to be so materially weakened with increasing length, as well as the inconvenience which very long columns present during the process of erection, the length of the columns between panel-points is usually fixed at from 18 to 25 feet, that of the top panel being generally the shortest on account of the greater convenience of erection at that height.

A graphic representation, or skeleton diagram of a tower intended to support the 20 × 42-ft. tank is shown on page 137. The distance between column-centres at the top is 14.14 ft., or the long chord of the 20-ft. circle. In a 70-ft. tower, where the columns are given an inclination of one in ten, the inclination of each column would be 7 ft., and the two 14 ft., which, added to the width apart at the top, makes 28.14 ft. as the distance apart, centre to centre, at the base of the columns. The stress applied to each column from the constant and variable load of the 20 × 42-ft. tank has been found to be 516 tons, hence each column must carry 129 tons; as the column has not only to carry the weight due to the constant and variable load of the tank, but also the weight of the tower, a column capable of bearing more than 129 tons must be designed or selected. This could be exactly determined by taking the panels successively, determining the weight of the members and adding the weight so found to the weight applied at the top of the column, and then designing a column of uniform cross-section, capable of safely sustaining this maximum weight; but in practice it is generally assumed, roughly, that the tower will weigh a little less than the tank, for towers and tanks of this approximate size; and as the tank was found to weigh 18 tons, one-fourth of this weight, if assumed for the tower, will make the entire weight to be sustained by each column 133 tons. Proceeding upon this hypothesis, and examining Carnegie's handbook, a suitable "Z"-bar column is found to be a 10-in. column, weigh-

ing, exclusive of rivets, 75.8 lbs. per linear foot and capable of sustaining a weight of 133.9 tons. The wind-stress may be considered as constant and as already provided for in the compression-members.

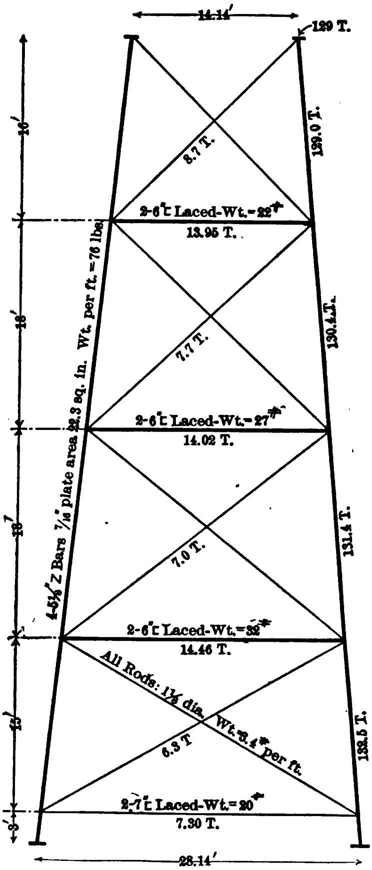


FIG. 17.—STRESS DIAGRAM.—70-FT. TOWER; 20 × 42-FT. TANK.

Determining the height of the panels as 16 ft. for the top and three of 18 ft., a total of 70 ft., the length of the horizontal compression-members at each panel-point and in-

tended to resist the thrust of the column due to its inclination, is manifestly 14.14 ft. plus twice the horizontal inclination. This last being one in ten, in 16 ft. the horizontal inclination is 1.6 ft. $\times$ 2 = 3.2 ft., which, added to 14.14 ft., gives 17.34 ft. as the length of the first horizontal compression-member. The remaining like members will have the same relative length to be determined in the same manner.

The stresses to which these members are subject are those of compression, from the weight of the load, and a bending-stress due to their individual weight. In the upper members, where the span is short, this last is not of much moment; but, as the span increases, the individual bending-stress becomes more important.

The inclination of the column being one in ten, one-tenth of the load is transferred to the horizontal member as compression-stress, and the remaining nine-tenths is distributed at the base of the column to the foundation. As it has been assumed that the weight which each column will sustain throughout its length is 133.9 tons, the first horizontal member, sustaining one-tenth of this load, must be designed to resist a compression-stress of 13.39 from the thrust of each column, or twice that for the two opposite columns between which the horizontal member will be sustained, or for 26.78 tons.

Connections.—In addition to this, the member must be of such strength that it will be able to restrain the stress applied as bending-stress. As this stress is due entirely to the individual weight of the member, acting through the centre of gravity, or midway between supports, it is obviously small for the shorter spans of the horizontal members, and for the shortest of these, the upper member, need not be considered, only the stress of compression received from the extraneous forces requiring attention.

As the compressive stresses may cause the member to fail

by flexure, this member must be designed to resist this force upon the principles of the column. The "channel" shape affords a convenient form for the individual member of such a column, and two such shapes, when riveted together back to back and laced top and bottom to prevent individual weakness and to further stiffen the entire section, make an inexpensive and serviceable compound shape.

In order to make the two independent channels act as one beam or unit, the theoretical length of the lacing has been found to agree with the formula $l = \frac{rL}{R}$, where

$l =$ length between bracing in feet,
$L =$ total length of strut in feet,
$r =$ *least* radius of gyration for single channel,
$R =$ least radius of gyration for entire section.

The standard handbooks give "l," of the formula, for a wide range of spans in feet. From Pencoyd, the length "l" of this spacing for the 17-ft. span under consideration is given at 4.38, approximately. In practice it is customary to greatly reduce this theoretical length, and this factor, divided by four, or 1.09 ft., may be taken as a safe distance for the spacing of lacing-bars.

In spacing channels, considering the fact that for columns the *least* radius of gyration is always taken, it is desirable to so space the channels that the same radius of gyration is had at either axis, but sometimes the conditions, or the character of connection required, prevents this arrangement. The manufacturers' handbooks give the spacing for identical radii for a number of standard channels.

Considering the connection of such horizontal member in relation to the standard Z-bar column, a spacing of 3.3 ins. of channels likely to be selected will afford an opportunity for a standard connection of the horizontal member to the column. From Pencoyd's handbook, two channels No. 61*C*,

when spaced as above, are found to have a radius of gyration of 2.29. The length of the horizontal member being 17.36, the ratio of length to radius is 7.6, and from the table Strength of Columns, the unit stress corresponding to this ratio is found to be 9746 lbs. per square inch of section. Neglecting the lacing, the area of the two channels is given in the handbook as $3.09 \times 2 = 6.18$ sq. ins.; then $S \times A$, or $9746 \times 6.18 = 60230$ lbs. or 30.1 tons as the safe load, and the weight of this section, $10.5 \times 2 = 21$ lbs.; with rivets and lacing-bars, say 22 lbs. per linear foot of member.

At each successive panel-point, the length of the member is due to the inclination of the opposite columns. The length of the horizontal member, taken 18 ft. from the base, may be calculated by taking either the top or bottom distance between columns; if the former, $70 - 18 = \frac{52}{10} = 5.2 \times 2 = 10.4 + 14.14 = 24.54$ ft.; from the bottom or base, 28.14, the inclination for 18 ft. is $1.8 \times 2 = 3.6$, which, taken from 28.14 gives the same, 24.54, as the length of the member 18 ft. from the base or 52 ft. from the top.

Taking the compressive stress as before 26.78 tons, owing to the increased length of span and consequent increased bending-moment, a heavier beam should doubtless be selected. In order to preserve the regularity of the section, take a heavier 6-in. channel.

Channel No. 63*C*, Pencoyd's handbook, is found to weigh 15.50 lbs., or say 32 lbs. per foot of built section. The maximum bending-moment being $\frac{WL}{8}$ or $\frac{785 \times 24.54}{8} = 2408$ lbs. or 1.2 tons, which, added to the constant 26.78, = 28 tons approximately.

The length of the horizontal member "l," divided by the radius 2.29, gives $\frac{l}{r}$, shown by table, a ratio of 10.8 =

S, or unit stress of 8180 lbs. The area of two No. 63C channels is given by Pencoyd as 9.12; hence $A \times S$, or 9.12 $\times$ 8180 = 37.35 tons, or a strength nearly 25 per cent. greater than required.

In long members of this class there is, however, another consideration besides the maximum safe strength, for such a section is apt to deflect beyond a permissible limit. This allowable limit is a most variable quantity, and has been given by Trautwine for bridge-deflection at .01 inch per foot of span. Such structures are subject to very variable loads. Tregold gives for the beams in buildings which sustain plastering an allowable limit of $\frac{1}{30}$ inch per foot of span. For such members in tower-work possibly $\frac{1}{20}$ inch per foot would not be radical, as a slight deflection would do no damage which would be the case where plastering must be preserved.

Taking as the allowable limit this constant and applying it to the length of the member under consideration, the permissible deflection for the 24.4 ft. would be 1.2 inches.

The deflection at the middle of such beams uniformly loaded may be theoretically determined by the formula $f = \frac{5WL^3}{348EI}$, or $f = \frac{WL^3}{76.8\,EI}$, where

W = weight uniformly distributed,
L = length of span in feet,
E = modulus of rupture,
I = moment of inertia.

To simplify this calculation, Carnegie's handbook furnishes a table based upon the above formula and using 28,000,000 as the modulus of rupture, and a fibre-stress indicated by CS and C^1S respectively, of 16,000 and 12,500. The greater fibre-stress may be accepted here.

In applying the table to find the deflection in 64ths of an inch for any length of span in feet, divide the coefficient in the following table, corresponding to the required length in feet, by the depth of the beam in inches.

DEFLECTION COEFFICIENTS FOR CARNEGIE'S SHAPES, GIVEN IN 64THS OF AN INCH.

Coefficient Index.	Distances Between Supports in Feet.											
	6	8	10	12	14	16	18	20	22	24	26	28
C^1S	38.1	67.8	105.9	152.5	207.6	271.2	343.2	423.7	512.7	610.2	716.1	830.5
CS	29.8	53.0	82.8	119.2	162.2	211.8	268.1	331.0	400.5	476.6	559.4	648.8

For the 24-ft. beam being considered, the coefficient, as taken from the above table, is found to be 610.2, while the depth of the beam is 6 ins.; hence $610.2 \div 6 = 101.7$ as the proper coefficient of the *maximum* safe bearing, but the beam being discussed is not subject to the maximum load by 25 per cent.; hence reducing the coefficient by that extent, we have 76.3, which is $\frac{76.3}{64}$, or 1.2 inches, the permissible deflection determined upon.

As the middle horizontal member of the 70-ft. tower is exactly proportional to the top and bottom member, without calculation, a mean section between these two would be safely applicable: this may be taken as the shape formed from two channels, No. 62 C, Pencoyd's shapes, and found to weigh 13 lbs., or say 27 lbs. for the entire built section.

While a sufficient number of horizontal compression-members have been designed to reduce the lengths of the individual column-sections to conservative and convenient lengths, it is considered good practice to introduce an additional horizontal member near the base of the structure in order to stiffen the tower—tieing it together, as it were—and to receive and transmit to the four columns near their base the wind-stresses which must be provided for and which will be subsequently considered. These stresses have been found for the structure under consideration to amount to 70 tons, and this stress may be considered as acting over four column-sections and the

four horizontal members which it is the usual practice to introduce, as has been said, near the base; therefore each of these members would receive $\frac{1}{8}$ of the entire stress, or 8.7 tons.

The length of this auxiliary horizontal member is found, as before, to be the distance between columns at that point, or if the member be introduced 3 ft. from the base, $28.14 - .6 = 27.54$.

As the length of span is great as compared with the stresses, the section designed for this member should be correspondingly deep, and may consist of an I beam or other rolled section, or of angles and web, or angles and lattice-web, or of the built channel-section as preferred for the other members.

Selecting a light 7-in. channel, No. 70 C, from Pencoyd's shapes, the weight is given at 9.75 lbs. for each channel, or say 20 lbs. for the compound shape. The bending-moment is therefore $\frac{562 \times 28.14}{8}$, or 1 ton.

This, added to the 8.7 tons wind-stress, makes approximately 10 tons to be resisted.

l being approximately 28, and r, from the table, 2.73, the ratio is 10.2, and from the table of ratios the value of the unit-stress S is found to be 8474; $A \times S$, or $5.72 \times 8474 = 24.23$ tons.

The stress being taken at 10 tons, the shape has an excess strength of about 40 per cent. For the length of span of 28 ft. the coefficient as taken from the table of coefficients is 830.5, which, divided by the depth, gives 118.6; reducing this 40 per cent, the deflection is found to be $\frac{71.2}{64}$ in., or 1.1 in., and as the allowable deflection is 1.2 in., the shape is within the permissible limit for deflection of such beams, and is a satisfactory compound member.

In the connection of the horizontal to the upright mem-

bers, standard angle-sections, limited in length by the style column and member designed, are usually employed. Such connections for a wide range of end-reactions are designed by the larger manufacturers, who have prepared details and bill of material of such connection-members for varying end-reactions. These connections are designed by the Carnegie Company with an allowable shearing-stress of 10,000 lbs. per sq. in., with a bearing-value of 20,000. lbs. on rivets and bolts, and an extreme fibre-stress of 16,000 lbs. per sq. in. for rolled shapes.

For suitable standard angle-connections for the members being considered and whose end-reaction may be taken at one-half the load, or, say 17 tons, connections corresponding to this reaction for the thickness of metal assumed for the columns can be found in Carnegie & Co.'s handbook.

Suitable bearings at both top and bottom of each column, termed the "capital" and "pedestal," respectively, must be designed to receive the imposed stresses and to transmit them to the foundations; but such capitals and pedestals, with bearing-plates of both cast iron and steel, with suitable angle-connections, and for a wide range of imposed load, are designed and manufactured by all of the larger steel-works, so that it is unnecessary here to introduce calculations or formulæ to determine suitable horizontal or end-connections.

Wind-bracing.—The effect of the wind upon the vertical projection of the tank-surface, which has been shown for the 20 × 42-ft. tank to amount to a possible stress of 70 tons, is exerted in diagonal lines from panel to panel, and represents the resultant of the horizontal and vertical forces. Diagonal rods should be supplied to resist this force, and these rods will act in tension. As the direction of the force is so extremely variable, two sets of rods should be used. These should be connected at panel-points and would therefore cross each other at some point of the panel-section. Through

these diagonals the stress applied to the top of the tower will be transmitted through its member and delivered to the foundations and bearing-soil. As the number of panels increase, the stress upon the diagonals becomes less, but in practice it is customary to design a rod capable of safely resisting the maximum stress near the top of the tower, and to make the other diagonals correspond. In the first panel, one-eighth of the load may be considered as being applied to the diagonal member, or 16,400 lbs.

Since one square inch of steel is considered to have a safe bearing-value of 15,000 lbs., the size of the rod would be $\frac{16,400}{15,000}$, or 1.0933 inches, about $1\frac{1}{8}$-inch area, or one inch round rod.

After erection the structure, when subject to its maximum load, may settle or become somewhat distorted, and to provide for an adjustment after such conditions, the diagonal rod is generally sheared into two lengths, and afterwards connected by means of "clevis" or "swivel" nuts, which, being threaded to correspond to threads cut upon the two adjacent ends of the rods, afford an easy method of tightening up the several members of the structure.

As one of the sheared ends will be "up-ended" and threaded, the size of the rod is generally made from $\frac{1}{8}$ to $\frac{1}{16}$ more than theoretically required; therefore $1\frac{1}{8}$ in. diameter rod of soft steel would be about the right design.

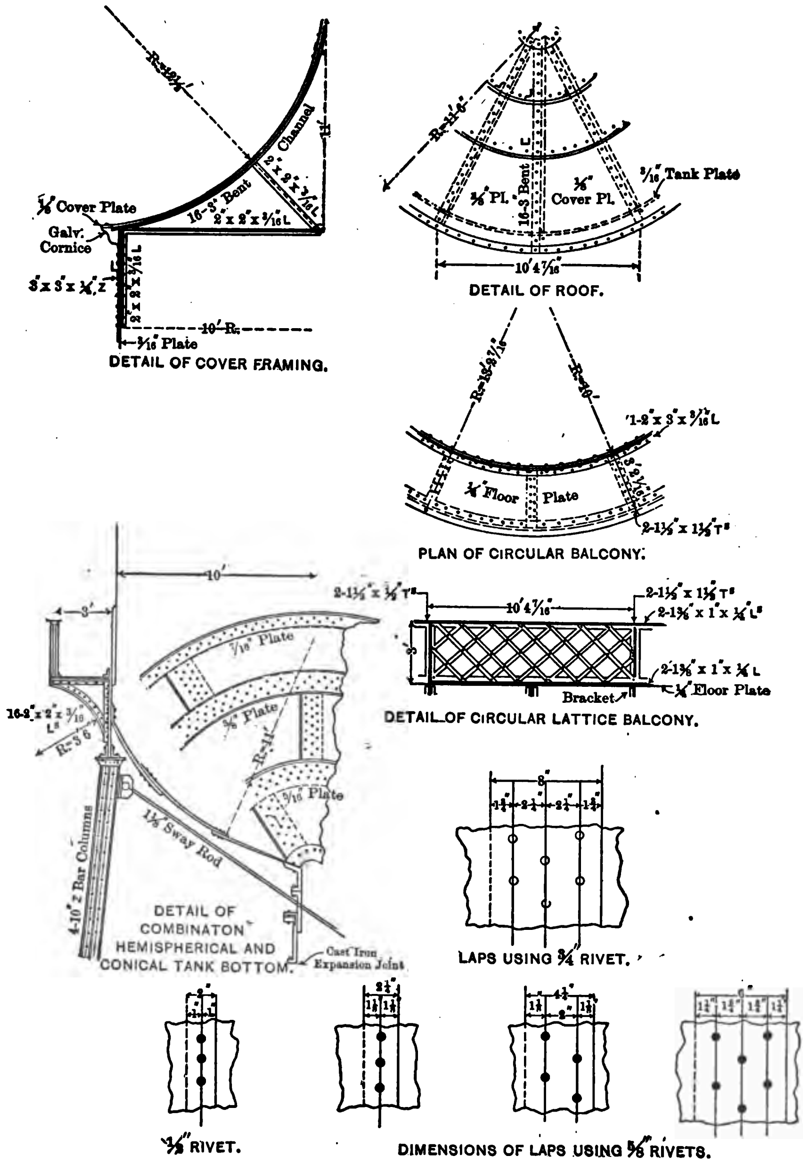
The preferable connection to the horizontal member is the "pin-connection," for which the rod is bent and welded so as to form an "eye," through which a steel pin is passed upon which the rod reacts, and the pin is secured by suitable steel plate riveted to the horizontal members near their panel-points.

This class of wind-bracing is in general use on account of its simplicity and strength as well as its inexpensive character,

but where the diagonals would prove objectionable, a type of bracing, called "portal bracing," is sometimes used. A most massive, ornamental, and effective example of this type of bracing is seen in the lower panel of the Eiffel tower, of Paris. Its general lack of utility for such structures as are being considered, as well as its increased cost for the same relative efficiency, precludes a further discussion of this type of bracing here.

In considering the stresses applied to the horizontal members, the stress to which they are subject in transferring the wind-stress as tensile stresses was not considered, and those members were designed only to resist the maximum stresses of compression and flexure.

Stability of Structure and Anchorage.—In investigating the stability of position of the structure, the same methods are used were employed as when the stand-pipe was being considered. The force of the wind, 70 tons, is exerted over a leverage equal to the height of the tower and half the height of the tank, or 91 ft.; then $70 \times 91 = 6370$ ft.-tons. The weight of the structure being taken at 18 tons, and its leverage being one-half the base, or 14.07, $18 \times 14.07 = 507$ ft.-tons. Selecting eight 3-in. steel rods, 7.07 sq. in. area each, and a working value of 15,000 lbs., or 7.5 tons, their combined holding-down force is 424 tons into their leverage $14.07 = 5966$ ft.-tons, and the sum of the two holding-down forces amount to 6473, or 103 tons more than is required to assure the equilibrium of the structure when the tank is empty.



TANK DETAILS.

FIG. 18.

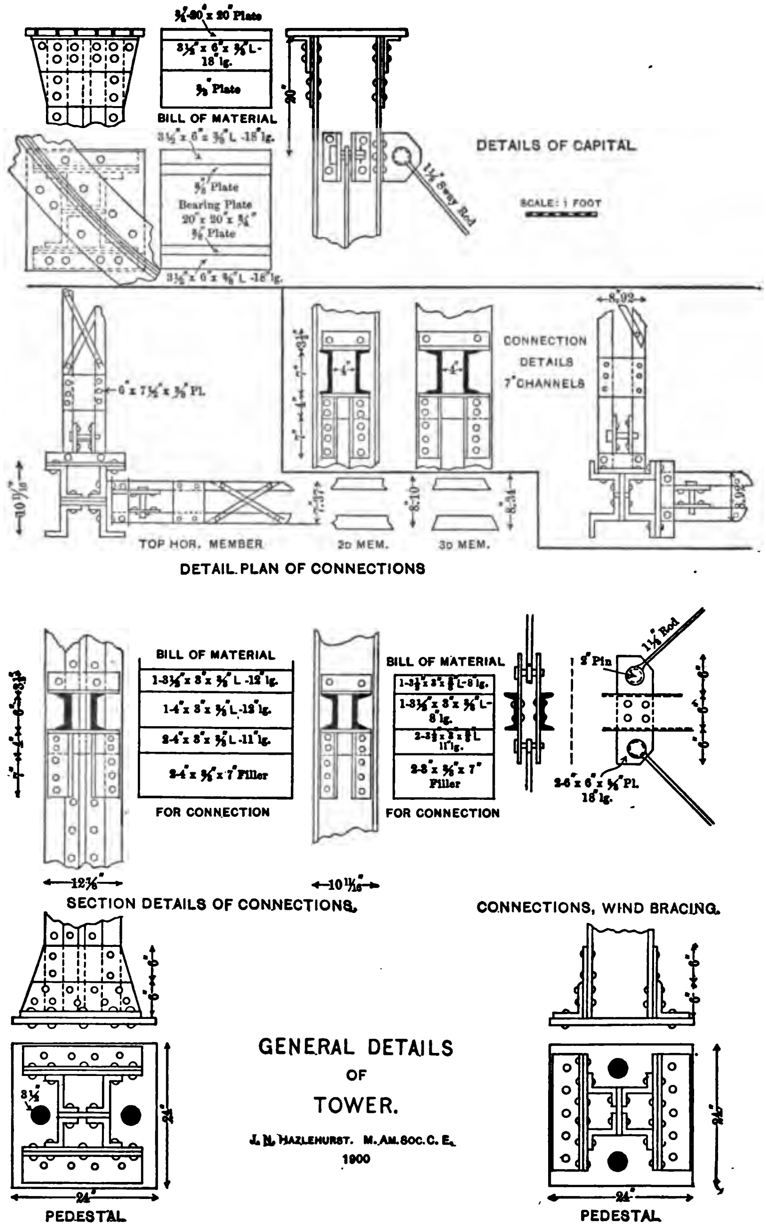

BILL OF MATERIAL
DETAILS OF CAPITAL
SCALE: 1 FOOT
CONNECTION
DETAILS
7" CHANNELS
TOP HOR. MEMBER
2D MEM.
3D MEM.
DETAIL PLAN OF CONNECTIONS
BILL OF MATERIAL
FOR CONNECTION
BILL OF MATERIAL
FOR CONNECTION
SECTION DETAILS OF CONNECTIONS.
CONNECTIONS, WIND BRACING.
GENERAL DETAILS
OF
TOWER.
J. N. HAZLEHURST. M. AM. SOC. C. E.
1900
PEDESTAL
PEDESTAL

FIG. 19.

CHAPTER IX.

FOUNDATIONS.

THE generic term, Foundations, comprehends both the soil and the materials upon which a structure is designed to rest; the line of demarcation or termination of the foundations and commencement of the substructure is variable, but in general the approximate ground-line is the limiting point. More exactly, every foundation may be regarded as having two components—the bearing-soil, or subfoundation, and the foundations proper, consisting of the materials intended to form a solid base for the superstructure.

The preparation of the natural soil for suitable subfoundations demands as wide a consideration and treatment as the wide difference of geological conditions, but in practice an intimate knowledge of the varying soil characteristics is not possible or hardly necessary, and it is considered sufficient to contrast the given soil with one or more of the more common formations whose qualities are determined from long experience. Such typical formations are rock, clay, gravel and sand, and alluvial soils.

Rock.—Discussing these in the order named, the best natural subfoundation is rock, in classification varying from the crystalline types to soft-deposit specimens, easily waterworn or subject to atmospheric disintegration, for experience has shown that any stone formation, well bedded, will safely sustain any load that may be imposed upon it by any masonry foundation, even for the largest structures.

Frequently the stone is not found in horizontal, continuous layers, but in seamy strata, offering a bearing-surface of more or less irregularity and composition. For the suitable preparation of such a subfoundation the overlying earthy matter and any decomposed or decayed stone must be removed to "bed-rock" or the solid layer, which is then blasted or sledged to a surface as nearly perpendicular to the pressure to be imposed as possible. Interstices or fissures of the rock should be filled with broken stone or concrete, and where the bearing will not be entirely upon stone, but upon contiguous earth, at such junction especial care should be taken to thoroughly compact the softer material or to remove it altogether, substituting broken stone, or preferably concrete, bedded as well as possible to the more unyielding natural stone by cutting the bed-stone in steps, or making some other effective union; otherwise unequal settlement, the result of unequal resistance, will result.

Clay.—Clay, when dry and likely to remain so, is an ordinary and excellent foundation, being easily excavated and having a safe bearing-value for ordinary structures; but clay is a treacherous material in that it so readily absorbs moisture, its seamy veins acting often as conduits for underground streams of varying magnitude. When clay absorbs water, its tendency is to swell and soften, and under such conditions, when confined, it exerts a material pressure upon the sides or bottoms of foundations, tending to bulge and crack them. When unconfined it spreads in every direction, oozing and squeezing from under the weight imposed and becoming unstable and uncertain in action. Exposed to the moisture of the air it becomes more or less saturated, and at low temperatures the mass freezes, expands, and disintegrates after a thaw, proving a most intractable material. From this fact, in preparing the subfoundations in such material, the excavations should extend well below the frost-line, and the ex-

posure of the foundation-pit to atmospheric influences should be as limited as possible, as a sudden rain may change a good foundation to a quagmire. Excavations in clay should be made immediately in advance of the actual masonry construction.

When wet, the bearing value of clay can be artificially increased and improved by incorporating with it, according to its plasticity, layers of sand or gravel, or both, or by spreading layers of concrete.

The tendency of the veins of the clay to transport water results in the discovery of springs of water of more or less volume in a number of foundation-pits, and these springs are a source of embarrassment and trouble, as they prevent the masonry from setting, or ooze or stream through the sides or bottom of the completed work.

Their treatment is largely a matter of personal experience, but the less troublesome varieties may be suppressed by plugging the water-bearing crevice with dry sand and cement, dry cement or concrete, either directly or upon some fibrous material, such as yarn, which will absorb the moisture until the cement has an opportunity to set, or upon some impervious material, such as tarred or oiled cloth; or by setting a tube over the aperture, and plastering about its foot with pipe-clay, or some plastic material, allowing the water to rise in the tube, or be drawn away through the tube while the masonry is being constructed. After the masonry has set the tube may be plugged with concrete below the face of the foundation, and then either cut off or withdrawn. These are only general suggestions, experience being the only safe guide in such emergencies.

Dry Sand.—Dry sand makes one of the best subfoundations if its status as such can be fully determined, for it is an almost incompressible body; is not affected by exposure to any extent, and its bearing power is therefore very great.

The size of the grains of sand may increase from very fine particles to coarse gravel; the coarser the grain, the better the foundation as a rule. Gravel and sand, when incorporated with a binder of clay, are cemented together to an extent which makes such a soil but little less valuable as a bearing material to the softer grades of rock, but where the grains of sand are fine, having no cohesion, the mass, when saturated with water, becomes semi-fluid, and is subject to hydraulic principles. Owing to its porosity and susceptibility to moisture, sand, like clay, is subject to the disintegrating effects of frost, and the foundation-pits should therefore be excavated below the liability of such exposure. Also like clay, having a capillary attraction for fluids, in sand foundations, springs are frequently encountered which should be treated as above suggested in the absence of more definite knowledge and experience. The same methods would apply for a weak clay foundation, such as spreading concrete over the area uncovered, is advisable to assist and to augment its bearing-surface, but frequently in such soils, as well as upon the clay variety, the bearing values are increased by removing a portion of the soft material and driving or jetting down short piles upon which stringers of wood are spiked, the spaces between rows being filled with concrete; sometimes the use of the stringers alone will be found sufficient in addition to the use of the concrete, which is compacted flush with the tops of the sills. Such construction is called "grillage," and is frequently used. Since timbers covered by water and removed from atmospheric oxidation have been proven to last for indefinite periods, such a foundation, where completely subject to saturation, is very effective and safe. In very soft sand, clay, or alluvial soils these methods are found effective, and in addition planking, making a floor for the foundations to be started upon, is spiked transversely

upon the tops of the stringers and over the concrete deposited between them.

Quicksand.—When sand is so completely saturated as to become fluid, it is termed "quicksand"; it has no peculiar qualities or inherent properties, but is generally given an individual classification.

Any saturated sand is "quick" when the upward pressures of the underground waters are sufficient to overcome the tendency of gravity to keep its particles at rest. Sand of coarse grains resists this upward tendency to a greater extent than the finer varieties; hence quicksand is usually a very fine grained sand, and from the fact that it must be found immersed in water, the constant friction of its particles moving upon each other grinds the sharp points and angles, until the grain becomes rounded or "water-worn," the usual condition of the grains of the so-called quicksand.

Increasing Bearing Values.—In very soft material, where the necessity of reenforcing the bearing-value of the soil is apparent, and where there exists an underlying soil of better material, the piles, when driven through the top soil, penetrating into the strata below, act as so many columns whose ultimate bearing is the crushing strength of the material of which the pile consists, but where there is no such lower soil the piles are supported in the soft material only by the friction of that material against their sides, and the determination of their safe bearing-value is more problematical. Rankine gives as a rule for the safe bearing of piles under this last condition the area of the head of the pile in inches by 200; thus a 12-in. pile, having an area of head of 78 sq. in., would give a safe bearing of 7.8 tons.

A simple rule frequently used for the safe bearing value of piles is one formulated by Major Sanders, of the U. S. Engineer Corps, from experiments made with common wooden piles at Ft. Delaware, and is as follows:

$$\text{Safe load in lbs.} = \frac{\text{Weight of hammer in lbs.} \times \text{fall in in.}}{8 \times \text{penetration at last blow}}.$$

Applying this to a 12-in. pile, driven with a 2240-lb. hammer, and penetrating $\frac{1}{2}$ in. at the last blow, the safe bearing is 101.3 tons.

This value, from the author's experience and opinion, is, on the contrary, too high, and a formula deduced by Trautwine more nearly answers the problem considered in the light of practical results. Trautwine's rule is:

$$\left.\begin{array}{c}\text{Extreme}\\ \text{load in}\\ \text{lbs.}\end{array}\right\} = \frac{\text{Cu. rt. of fall in ft.} \times \text{wt. of hammer in lbs.} \times .023}{\text{Last sinking in inches} + 1}.$$

Taking the same constants as above, the *extreme* load is 106.6 tons. In order to arrive at a safe load, some factor of safety must be used, and if 2 is taken, then the safe load becomes 53.3 tons, which is considered about right in practice. In very soft ground a larger factor should be used, from 4 to 6 being the practice.

In piles supported by the friction along their sides, the ultimate value of that friction is estimated at from .2 to 1 ton per square foot of bearing for each foot of length, depending upon the soil characteristics. In silt or wet river-mud, when driven three feet apart, the possible value of friction upon unbarked piles is .5 tons per foot length. In New Orleans, where the soil is a saturated alluvial for 900 feet depth, piling is used for all building foundations where much weight is to be imposed. In some of the larger buildings, even with this addition to the bearing-values, considerable settlement has been observed. A foundation designed for a stand-pipe, 13 × 100 ft., in that locality, consisted of 100 piles, driven an average of 60 ft. deep, and spaced 2 ft. in both directions. The piles were of unbarked cypress, aver-

aging .5 cu. ft. per foot length. Although continuing to penetrate under the blows of the hammer considerably more than ½ in., the piling was stopped at 60 ft., upon the theory that the frictional resistance through that depth would equal .5 ton per foot of pile length or 3000 tons for the 100 piles. Assuming a factor of safety of 5, the safe bearing was determined at 600 tons, which represented the total weight of the tank, water, wind-stresses, and foundations.

No observable settlement in this foundation has taken place in several years. The piles were sawn and capped; the longitudinal spaces were filled with concrete flush to the top of stringers, and the grillage floored, all timber being below the point of saturation of the soil. All earth foundations must yield somewhat, but this is not important in the case of isolated structures such as stand-pipes and the like, provided the settlement is gradual and uniform, and not of radical extent.

The following table represents the safe values of ordinary soils according to Prof. Ira O. Baker:

SAFE BEARING-VALUE OF SOILS.

Kind of Material.	Safe Bearing-power in tons per sq. ft.	
	Max.	Min.
Rock, the hardest, in thick layers, in native bed........	..	200
" the softest, easily worn by water or exposure to the weather..	..	18
Clay, in thick beds, always dry........................	6	4
" " " " moderately dry....................	4	2
" soft beds..	2	1
Gravel and coarse sand, well cemented................	10	8
Sand, compact and well cemented........................	6	4
" clean and dry..	4	2
Quicksand and alluvial soils............................	1	0.5

Stone Masonry.—The requirements for a serviceable foundation building stone are, in the main, that it shall be

hard, tough, close-grained and durable. Upon its closeness of grain and non-porosity depend its non-absorbent properties, without which the stone is likely to disintegrate along its layers. A stone with a granular texture is likely to crumble in weathering to a greater extent than one with a crystalline formation. Before determining upon a building stone, and where a choice is possible, investigation as to its possible usefulness for the particular service required should be made by an examination of the effects of exposure and service upon like stone in any old structure, or by an examination of the quarry, where the effects of weathering and decomposition should be carefully observed, noting whether the stone has disintegrated to an appreciable extent, or has corroded, or whether the old lines of fracture remain sharp and fresh. Where a new quarry is to be opened, and there is any doubt as to the character of the stone, it should be subjected to artificial tests such as crushing, abrasion, etc.

The more common and serviceable building stones are granite, limestone and sandstone, in their several varieties. The cost of quarrying such stone will depend upon such factors as the wages of the quarrymen, the mechanical facilities for such work, as well as the amount of "stripping" necessary, and other items likely to affect their cost. Roughly, stone can be quarried at from 40 to 80 cts. per cubic yard, varying in different localities and unlike conditions.

Stone masonry is of various classes, but for such foundation work as the foundations for stand-pipes, it may be assumed that it will be either ashlar, range rubble, or rubble, laid in cement-mortar.

Ashlar is the highest grade of masonry; it is squared dimension-stone, cut with varying degrees of nicety, and is consequently considered as first class, second class, etc., owing to the finish required.

Owing to the care necessary for its preparation, it would

hardly be employed, owing to its cost, upon any portion of a foundation for a stand-pipe except possibly the first course immediately below the superstructure, where such course is exposed. Frequently the cut stone is used only as a belt upon the outer perimeter of the foundations, the interior or core being "backed up" with rough rubble masonry, well flushed and levelled with cement. This last type of masonry consists of rubble proper and range rubble masonry; the former being stone of almost any dimension, roughly sledged for use, and bedded in cement without regard to horizontal jointing; range rubble requires that the stone shall be laid to a rough line horizontally; the first of these distinctions of rubble masonry is generally used below the ground-line and for the core of the foundations, while the range rubble is employed for the exposed surfaces up to the first course under the structure, which is frequently of ashlar finish. As with the quarrying, the local conditions modify the cost of all masonry work, but roughly the following will give an idea of the relative value of several masonry classifications:

First-class ashlar	$12.00	to	$15.00	C. Y.
Coursed rubble	4.00	"	6.00	"
Rough rubble	3.00	"	5.00	"
Concrete—1 part Port. cement, 2 sand, 4 broken stone	4.00	"	6.00	"
Ordinary brick masonry—cement mortar	5.00	"	8.00	"

In stone masonry, Rankine's general rule, modified to suit particular conditions and individual ideas, is largely used and is as follows:

RANKINE'S RULE.

I. Build the masonry as far as possible in a series of courses, perpendicular, or as nearly so as possible, to the

direction of the pressure which they have to bear; and by breaking joints avoid all long continuous joints parallel to that pressure.

II. Use the largest stones for the foundation course.

III. Lay all stones which consist of layers in such manner that the principal pressure which they may have to bear shall act in a direction perpendicular, or as nearly as possible, to the direction of the layers. This is called *laying the stone on its natural bed*, and is of primary importance for strength and durability.

IV. Moisten the surface of dry and porous stones before bedding them, in order that the mortar may not be dried too fast and reduced to powder by the stone absorbing its moisture.

V. Fill all parts of every joint, and all spaces between the stones, with mortar, taking care at the same time that such spaces shall be as small as possible."

From various authorities the following table has been compiled:

SAFE BEARING-VALUE OF MASONRY AND MODULUS OF RUPTURE OF MATERIALS.

	Mod. of Rupture per Sq. In.	Crushing Strength per Sq. Ft., in Tons.
Granite	1800	75
Limestone, common varieties	1500	62.5
Oolitic limestone	2338	17.5
Sandstone (brownstone)	2160	60.0
Concrete, 1 month, 1 part Port. cement, 2 parts sand, and 4 parts broken stone	150	7.0
Brick laid in Port. cement, 1 to 2 mortar	800	10.0
" " " Rose'le " 1 to 2 mortar	800	8.0

Brick Masonry.—There is no generally recognized manufacturers' standard brick, the general character and dimensions varying considerably in different localities, but an average size is $8\frac{1}{2}'' \times 4'' \times 2\frac{1}{2}''$; such brick, when dry, will weigh about 5 pounds each, and in rough reckoning 500 such brick are

estimated as making a cubic yard of masonry, which weighs approximately 1.2 tons. With such brick an ordinary mason, with one helper, will lay 2000 in foundations. In such work, below the surface, the brick can be rapidly placed in courses and then grouted in by "slushing" cement-mortar over the surface, which fills the interstices and makes a bed for the succeeding course; in such foundations bats may be used in moderate numbers. At the ground-line more care is taken, and the brick are laid to a horizontal line, those forming the face being carefully laid, and the mortar-joints, which should not be over ¼ in. thick, are "struck" and neatly pointed. A good foundation-brick should be of close clay texture, well made, hard, and carefully burned. When two such brick are struck smartly together they should give a clear, metallic ring. Foundation-brick should not absorb more than about 7 per cent. of their weight of water after immersion for 24 hours. The color of a brick is no index of its qualities, although where the clay soil contains oxide of iron the color of the brick after burning will be red, and a good foundation-brick will be a "cherry-red." Obviously, the bearing-value of brick varies with the texture of the material, its care in making and burning, and the skill with which it is erected into masonry when bonded with a suitable mortar. As shown by table, page 158, the safe bearing-value of brick masonry, in cement-mortar, is taken at from 8 to 10 tons per square foot, and experience has shown this to be a safe and conservative value. Numerous tests have been made upon piers erected under different conditions by the United States government and individuals, but it is doubtful whether such experiments are of much practical value.

While in no wise conclusive, the failure of a brick pier and the collapse and total destruction of a tower and tank designed by the author, gives an opportunity to present certain facts in that connection which may assist in throwing

some light upon the ultimate resistance of brick masonry under normal and actual conditions.

Below the ground-surface, with a bearing-soil of good, stiff clay, four piers of 6 ft. base and 2 ft. square tops, constructed of sound, hard-burned Georgia clay, laid in a mortar consisting of 1 part Belgian cement and 2 parts sharp road-sand, and into each of which two anchor-rods $1\frac{1}{4}$ ins. diameter with $12 \times \frac{3}{8}$-in. boiler-plate washers had been inserted, had been constructed for the support of a 13-ft. diameter by 25-ft. high steel water-tank, supported by a four-column tower, 40 ft. in height. Upon the detail drawings a 24×24-in. cap was shown, but owing to a misunderstanding as to who was to furnish this bearing-plate, the cap was not provided. A delay in securing the necessary anchor-rods from the manufacturer resulted in the purchase by the assistant engineer of a set of $1\frac{1}{4}$-in., 5-ft. rods, supplied with the 12 ins. square boiler-plate washers. Later, when the original rods were received, accompanying them was a set of 12×18-in. washers, which, through the carelessness and ignorance of the assistant engineer and the erecting foreman, were set on top of the foundations to serve as bearing-plates for the tower. The piers were completed exactly 45 days before the final test, at which time the tank was filled within 2 feet of its top, when the foundations gave way and the whole structure failed.

The weight of the material was 28,000 lbs., the weight of the water at $62\frac{1}{2}$ lbs. per cu. ft. was 192,000 lbs., the approximate weight of each pier was 9,000 lbs. At the time of the failure there was no wind blowing, so that the total weight applied as compression was 256,000 lbs., or 128.0 tons. With the 24×24-in. cap specified, the bearing upon the masonry would have been 8 tons per square foot.

Under the conditions at the initial moment of failure, the entire weight of the tank and load, amounting to 110 tons,

was concentrated upon the 12 × 18-in. washer used as a cap, and this downward tendency was resisted by the holding-down power of three 12 × 12-in. washers in the three other piers, or a total of 648 square inches. Investigations made after the failure show that the excessive weight caused the column to puncture the pier through its entire length, coring out and completely crushing the brickwork contained between the two anchor-rods, representing an area of about 14 or 15 ins. square. Immediately below this core, the brick footings were intact, and a solid section 14 × 15 ins. was buried or driven into the bearing-soil of clay. The masonry around the column, which had penetrated into the solid masonry about 3½ feet, was not crushed, but was ruptured radially along the cement-mortar joints. Before the failure the piers were tested both with an engineer's and mason's spirit-level, and were checked as being truly horizontal and of the same height. The resistance offered by the subfoundation-soil to the penetration of the 14 × 15-in. section of footing course might be considered as amounting to 10 tons, and to that extent reducing the weight applied as downward pressure at the initial moment of rupture; under this supposition, the ultimate bearing of the masonry was 100 tons ÷ 4.5 = 22.2 tons. Although 45 days had elapsed since the completion of the piers, the cement-mortar in the centre of the pier had not fully hardened and was rather crubbly, although that exposed to the atmosphere nearer the surface was well set and very tenacious. After the failure the piers were torn away and new foundations, built upon the original dimensions, were substituted, and upon a 24 × 24-in. cast-iron cap the structure was built according to original design and has been perfectly stable during the past two years.

Concrete Foundations.—In general engineering work, concrete is a most useful material. It is formed of broken stone from ¾ in. to 2 ins. in longest diameter, of gravel, broken brick, shells, etc., the voids of the mass being filled with

cement-mortar of various proportions, depending upon the ratio of voids of the material. In practice, a good concrete can be made with one part of cement, two parts sand, and four parts broken material. In foundation-work, a good grade of Portland cement, sharp sand, and *clean* stone should be insisted upon. The volume of water used to incorporate the mass is the subject of never-ceasing discussion amongst the engineering fraternity, but in the author's practice a good concrete has been made by so dampening the mixture that after being deposited and rammed, a slight appearance of *water* upon the surface is all that is necessary. Concrete for small foundations is usually mixed by hand, upon a 12 × 12-ft. frame or light platform, the ingredients being placed conveniently. A proportion of sand, by measure, is first spread over the board into which is dumped the specified proportion of cement, and the two components thoroughly incorporated by the workmen with their shovels; spreading this mixture so that it shall be somewhat higher along the outer edges of the mixing-board, water is sprayed from a small hose upon the mass, which is quickly turned with shovels until every particle has been completely incorporated. Into this liquid paste the proper proportions of stone, after a drenching, are added, and quickly turned by the laborers until each particle of stone has been coated with the mortar. The concrete is then carefully deposited by the shovels of the workmen, in layers from 3 to 6 ins. thick, into the foundations. Such mixing and spreading by hand will cost approximately 60 cts. per cubic yard; the cost of the concrete will depend upon varying conditions, and will range from $4.00 to $6.00 per cubic yard in place.

Maximum Pressures.—The action of the wind upon the cylindrical surface of a tank and the application of that force as pressure upon the base has been previously explained. The normal pressure due to the load is the weight divided by

the area, and the maximum pressure to be transferred to the subfoundations will consist both of the normal and variable pressures. From the principles of resistance of materials, previously explained, the "live load" or variable pressure due to the wind can be found from the formula

$$\text{Wind pressure} = \frac{Ml}{2I};$$

and the maximum pressure will therefore be

$$\text{Max. pressure } \frac{W}{A} + \frac{Ml}{2I};$$

where M = moment of the wind;
l = the leverage at the base;
I = moment of inertia of the shape.

Where it becomes necessary to extend the base of a foundation in order not to overload the bearing soil, the foundations will extend in regular courses, and the safe projection of the successive courses will depend upon the pressure applied as force and the resisting quality of the material of which the courses are composed.

The theory of this action and resistance is given by Prof. Ira O. Baker, in "A Treatise of Masonry Construction," and is as follows:

"The area of the foundation having been determined and its centre having been located with reference to the axis of the load, the next step is to determine how much narrower each footing-course may be than the one next below it. The projecting part of the footing rests as a beam fixed at one end and uniformly loaded. The load is the pressure on the earth or on the course below. The set-off of such a course depends upon the amount of the pressure, the transverse strength of the material, and the thickness of the course.

"To deduce a formula for the relation between these quantities,

let $P =$ the pressure in tons per square foot at the bottom of the footing-course under consideration;

$R =$ the modulus of rupture of the material in pounds per square inch;

$p =$ the greatest possible projection of the footing-course in inches;

$t =$ the thickness of the footing-course in inches.

"The part of the footing-course that projects beyond the one above it is a cantilever beam uniformly loaded. From the principles of the resistance of materials we know that the upward pressure of the earth against the part that projects *multiplied by* one-half of the length of the projection is *equal to* the continued product of one-sixth of the modulus of rupture of the material, the breadth of the footing-course, and the square of the thickness. Expressing this relation in the above nomenclature and reducing, we get the formula

$$p = t\sqrt{\frac{R}{41.6P}}, \text{ or with sufficient accuracy, } p = \tfrac{1}{6}t\sqrt{\frac{R}{P}}.\text{"}$$

This represents a theoretical maximum set-off for the masonry courses, but in practice, as has been explained, it is usual to reduce this theoretical maximum allowance by a suitable factor of safety, and, in this particular, a factor of safety of 5 to 10 is customary and considered a safe practice.

In addition to the forces acting upon the foundation-soil, the material of which the actual substructure will consist adds its weight to the other forces as pressure upon the subfoundations, and therefore a general knowledge of the weight of different varieties of masonry is necessary. On the following page will be found a table giving the approximate weights of

the several building materials most generally used in stand-pipe foundation-work and compiled from various recognized authorities:

WEIGHT OF MASONRY IN TONS PER CUBIC YARD.

Weight of granite or limestone, dressed throughout (ashlar)	2.2 tons.
" " " " " rough rubble	1.8 "
" " sandstone, ashlar	1.9 "
" " " " rubble	1.6 "
Brick masonry, medium work	1.6 "
Ordinary concrete	1.4 "

Designing Foundations, Including Anchorage and Capping.—To design a suitable foundation for a particular structure the normal weight must first be determined or assumed.

Considering a proper design for a stand-pipe 24 ft. dia. $\times$ 120 ft. in height, and whose actual weight was considered as 80 tons, and whose dimensions would add 1696 tons as the weight of the water, or a total of 1776, and which weight should be first considered as acting over a base equal to the area of the structure, or 452.4 sq. ft., or with a unit-stress equal to $\frac{W}{A}$ or $\frac{1776}{452.4} = 3.9$ tons per sq. ft.

Neglecting for the moment the weight of the foundations, and which can only be obtained after a suitable design has been determined upon, to secure the maximum pressures per unit of bearing-surface, in addition to the normal weight divided by the area, there must be added the forces due to flexure or to the effect of the wind upon the cylindrical sides of the stand-pipe and as applied through its leverage to the base and over the area to be covered by the foundations.

Substituting the proper values in the formula $\frac{Ml}{2I}$, or for a cylindrical figure 24 ft. dia. $\times$ 120 ft. in height, and taking 30 lbs. per sq. ft. of diametral surface, as has been explained, as the action of the wind upon the sides of the cylinder, the force exerted by this variable quantity is

$$\frac{2592 \times 12}{32572} = .9 \text{ tons per sq. ft.,}$$

while

$$\frac{W}{A} = 3.9,$$

or a total of.......... $\overline{4.8}$ tons per sq. ft. of bearing.

If, after suitable tests, the soil was considered capable of sustaining this load, the foundations could be carried vertically, and directly under the structure without any "spread," and in such a case only a sufficiency of masonry need be provided to secure a proper anchorage, and intended simply to resist the overturning moment, without increasing the bearing-area. In such a case, the stability of the structure having been determined by the principle of moments, as has been explained, and a sufficient number of rods provided to prevent the overturning of the structure, the holding-down power of these rods must be secured by designing for each rod a "washer" or bearing-surface, upon which a sufficient load could be imposed in the shape of masonry as to resist the effects of the horizontal action of the wind tending to overturn the structure at its toe.

Now this overturning moment has been found to be approximately 2592 ft.-tons, while the resisting moment, being 80 tons of material, multiplied by its leverage, 12 ft., is 960 ft.-tons, leaving an excess overturning moment of 1632 ft.-tons which must be resisted by designing some form of anchorage.

The load which the anchorage is required to resist is found by dividing the excess, 1632 ft.-tons, by the leverage of the anchorage, in this case say 12 ft.; hence the combined strength of the anchorage to prevent overturning is 136 tons, and the strength required of each rod is found by dividing this product by the number of rods.

Since the area of a circle represented by the base, 24 ft. diameter, is 452.4 sq. ft. for ordinary brick masonry whose weight is 1.6 tons per cubic yard, each vertical foot of foundation weighs 26.88 tons, therefore $\frac{1632}{26.88} = 6$ ft. as the height of the substructure.

As has been explained, the anchorage consists usually of iron or steel rods set in the masonry and bolted to some external shapes riveted to the superstructure. Such rods receive their holding-down or resisting stresses from flat washers supported by the bolt-head of the rod and acting against the masonry above, and must be designed of size and strength sufficient to prevent their being bent downward or broken off, and with a surface sufficiently broad to prevent the masonry from giving way, thereby permitting the washer and bolt to crush the masonry and pull through, and their bearing-area must therefore be such as to distribute the applied load over a sufficient portion of the masonry to prevent overloading and crushing.

If ten rods and washers were provided as anchorage and with a leverage of 12.5 ft., each rod would bear $\frac{1}{10}$ of the total applied stress, in this case $\frac{1}{10}$ of 1632, or 163.2 ft.-tons, and this divided by their leverage, 12.5 ft., each rod and washer must be designed to resist 13 tons pressure, or a total stress of 26,000 lbs.

Such washers are usually of cast iron with a unit maximum shear value of 20,000 lbs. per sq. in.

The safe bearing-value of masonry as taken from the table being approximately 10 tons per sq. ft. or 144 sq. in., for brick, the area of the washer to resist the applied stress would be $\frac{13 \times 144}{10}$, or 187.2 sq. in.; and if a circular washer were used, its diameter would be about 15 to 16 in. and the unit-stress 140 lbs. per sq. in. over the surface. The

transverse strength of such a plate or washer depends upon its thickness, and an exact formula is difficult to arrive at, but that used by Kidder is probably upon the safe side, and is as follows:

$$\text{Thickness of plate in inches} = \sqrt{\frac{W \times P^2}{1600}},$$

where W is the unit load per square inch—in the present case 140 lbs.; P, the projection of the edge of the plate beyond the rod, in this case say 6.5 in. Substituting these values in the formula, the thickness of the cast-iron plate or washer is a little less than 2 in. at its thickest part next the rod.

As a rule, the bearing-value of the soil will seldom be considered safe for a load as great as that considered above, and the bearing-value of the soil must be increased by spreading the foundations over a greater area.

In order to consider such a condition, assume that the bearing-value of the soil is not over 2 tons per sq. ft. of surface, and that the same conditions exist as were considered in the preceding example. Let the safe bearing 2 tons be represented by B, and $B = \frac{W}{A}$; then $A = \frac{W}{B}$. Let A be the total area and W the total load.

The total constant weight of the tank and water was found to be 1776 tons; the wind-pressure, approximately 1 ton per sq. ft., exerted over an area of 452 sq. ft., adds 452 tons; while the weight of the masonry was estimated at about 27 tons per vertical foot, and for 6 feet amounts to 162 tons, or a total, W, of 2390 tons. Substituting this value for W in the formula $A = \frac{W}{B}$, the required area of base is about 39 feet; but spreading the base increases the weight of the foundations, therefore some greater diameter must be selected

and determined by experiment. In order to allow for a marginal projection for the anchor-rods, the perimeter of the upper plane of a conic frustum, which is a suitable form for the foundation of a stand-pipe, might be that for a 27-foot-diameter circle, which would allow an annular space of 18 ins. around the 24-ft.-diameter tank. If such a conic section is considered in cross-section, the lower base projects beyond the upper with a length equal to half the difference on either side, and this projection, representing the spread of the masonry, is secured by offsets in the masonry courses, the number and height of such offsets determining the height of the figure or foundations.

As shown, the maximum theoretical projection may be determined from the formula $p = \frac{1}{6}t\sqrt{\frac{R}{P}}$; and if the masonry is in courses of brick whose thickness, t, is 2.5 in., with a modulus of rupture R, according to the table, of 800 lbs., and a pressure at the base, P, of 2 tons, substituting these values in the formula, the *maximum* theoretical offset is 8.3 in., to be reduced by the use of a suitable factor of safety.

The maximum *safe* projection of brick in single courses, as determined by practice and ordinance in many cities, is $\frac{1}{4}$ the length of a single brick, or a fraction over 2 ins., or a factor of safety, using the formula above, of 4, which, having been used in designing throughout, will be continued in foundation work where the masonry is an almost solid monolith.

For experiment, selecting a 44-ft.-diameter circle as the required base, the projection, being the difference between that and the 27-ft. diameter, or 17 ft., the projection on either side is 102 ins., and the projection allowed for each course being 2 ins., there are 51 projections, whose thickness being 2.5 inches, the height of the foundations is 10.6 feet. From these quantities the exact total weight can be determined, and is as follows:

Constant weight of tank and water	1776 tons
Wind-pressure exerted over foundation-base	517 "
Weight of masonry	634 "
Total applied weight and stress	2927 tons

Then if B, or allowable bearing-value, $= W$, total weight, or 2927, $\div A$, total area, or 1520, the actual bearing under the given conditions is 1.92 tons, or a bearing slightly less than the assumed safe bearing-value of the soil.

In designing the foundations for a tower and tank, the same formulæ and methods are employed. To determine the wind-stresses, however, the moment of inertia I is, of course, that for a rectangle instead of for a circle, when 4 columns are used.

The supporting columns of a tower should be provided with some form of cap to distribute the weights and stresses safely over the masonry piers, and with the smaller structures cast-iron caps are usually designed, while for the more imposing superstructures stone caps are generally employed. In practice, the limiting admissible unit-stresses over the masonry is generally taken at 100 pounds per square inch of bearing-surface, and in the case where a cast-iron cap is deemed advisable, the formula previously given is a convenient and safe rule for determining the thickness of the bed-plate. Thus, if the applied pressure from one of the columns is 32 tons, or 64,000 lbs., the bearing-plate should be approximately 25.3×25.3 to transfer this weight to the foundations with a unit-stress not exceeding 100 lbs. per sq. inch of surface; if a 6-in. Z-bar column with a foot-plate 12×12 ins. were used, the projection p on either side would be 6.6 inches. Substituting these values in the formula, thickness of plate in inches $= \sqrt{\dfrac{100 \times 43.5(P^2)}{1600}} = 1\frac{11}{16}$ ins.

Where it is decided to use a stone, it should be of good, sound, and close texture, preferably of granite, the bearing-

surfaces, at least, to be "patent-hammer" dressed, and it is considered good practice to limit the least thickness of the stone to $\frac{3}{8}$ of its length.

If the column delivers to such a cap-stone 132.5 tons weight, or 265,000 lbs., with a unit bearing-value of 100 lbs., the stone would be approximately 52 inches square and the limiting depth being $\frac{3}{8}$ its length, 20 inches would probably be taken as the least depth. The bearing-surfaces of the stone should be truly horizontal when set; the depths should exactly correspond, and rod-holes for the anchorage should be carefully drilled from templets.

CHAPTER X.

PAINTING.

Discussion.—A lay-writer has clearly defined the science of engineering as "Common sense, directed by theory and practice, to works of construction," and he might have added "whose comparative permanency was a prime consideration."

This last, as a desideratum, it seems is frequently omitted by the engineer as well, and content with selecting materials and designing members, scant consideration is given to the necessity for effectually preserving the works of his creation when once they have been completed and tested.

Engineers' specifications for the protective coating for iron or steel too often exhibit a variability which permits almost anything in the nature of paint to be applied as a preservative, provided it is not too expensive, dries quickly, covers the ordinary stains, and for a time looks well.

A more satisfactory explanation is to attribute this neglect to a lack of knowledge rather than to a lack of interest, which is more to be condoned in view of the absolute diversity of opinion of those recognized as authorities as to what constitutes the best method of protecting metallic structures from corrosion and decay, and the further fact that possibly in the practice of the individual he has developed the anomalous idea that the cheapest paints have at times evinced, in actual use, superior qualities to scientifically correct and high-priced compounds.

A communication was received a short time since from a

well-known authority upon the manufacture and properties of structural steel in reply to a request for his opinion as to the best protective coating for steel, in which he says that he "knew no more about it than the average engineer. This is equivalent to saying that I know nothing, for there seems to be a radical difference of opinion on this question, and one engineer will claim that one kind of material is the very best thing that can possibly be used, and the next man will claim that it is the very worst. It reminds me of the investigation made by the *L. A. W. Bulletin* on the "Best Lubricant for a Bicycle." They published their conclusions, which ran about as follows:

1. Vaseline is the best lubricant.
2. Vaseline is no earthly good."

Considering the immense and increasing amounts of iron and steel used annually as structural materials for marine work, buildings, trusses, bridges and the like, and the limited and conflicting knowledge of the best methods of protection, it is surprising that accidents are not more frequent and serious, and that coroners' juries are not more often called upon to render similar verdicts to that given in investigating a celebrated bridge-failure and accident, where the jury found that "All went in, none came out, and there is nothing to sit on."

Iron-rust.—Although the best methods of preventing corrosion may be involved in uncertainty and dispute, the cause of the destruction of ferric members seems to be fairly well established and it is a generally accepted scientific theory that, primarily, rust or metallic corrosion is the effect of a chemical combination of carbonic acid gas, oxygen, and water with metallic iron, producing ferric oxide or iron-rust which, once affected, continues with great rapidity through both chemical and galvanic action.

It has been shown by frequent experiment that carbonic acid gas and oxygen, together or separately, will not pro-

duce the phenomenon of rusting until water is added to complete the compound. Fresh water alone, when free from acids or organic impurities, has been found to have but little effect upon submerged plates of bright iron or steel, but where the plate is entirely or intermittently immersed in salt water, the salt water, taking the iron oxides into solution, removes the oxides and exposes fresh metallic surfaces to attack, also setting up a voltaic action upon ferric bodies.

Structural work is generally exposed only to atmospheric action, the atmosphere being sometimes charged with salt-sea vapors, and always with some moisture, in addition to the three universal components—nitrogen, oxygen, and carbonic acid gas—in the presence of which the destruction of ferric members is sure; the intensity and extent of this action being directly dependent upon the quantities of each element entering into the chemical action.

Chemical and Galvanic Action.—The chemical reaction in such cases is the setting free of the hydrogen of the water, its oxygen, uniting with the carbonic acid and metal, forming ferrous carbonate, which again combining with the oxygen of the water or atmosphere; is decomposed into ferric oxide and carbonic acid gas, the latter passing off, leaving the sesqui-oxide of iron to absorb and condense water, becoming the hydrated sesqui-oxide of iron whose symbol is $2(Fe^2 O^3)3H_2O$, ordinarily known as iron-rust.

It is a familiar fact that bright iron or steel may, under favorable conditions, be kept unprotected free from rust for a considerable time, but that when once the process of rusting commences, the rust specs, as centres of corrosion, rapidly spread until the entire metallic surface becomes covered with a sheet of rust. The chemical explanation of this progressive action when rusting has once commenced is, that during the decomposition by oxidation of the ferrous carbonate to ferric hydrate, the entire amount of carbonic acid is not given off,

and acts upon the new surfaces of the metallic iron, and owing to the porous and hygroscopic character of the rust crust, only small quantities of oxygen and moisture are necessary to indefinitely continue the process, the hydrated oxide giving no protection to the underlying metal. The capacity of rust for absorbing and condensing moisture and oxygen is enormous, and it has been proved that iron-rust will absorb as much as 27 gallons of oxygen-gas in making one pound of rust.

It seems beside the strictly chemical action, there is a galvanic effect which augments the work of corrosion and destruction when once begun; for it has been shown that the oxides of any metal are electro-negative to the metal itself, and that in ferric oxide a voltaic action is set up in its fibres and surfaces in contact by thermo-electric currents due to changes of temperature of the body; further, that the contact of such products as iron and steel is sufficient to set up such action, the result being a pitting and corrosion of the material, now technically known as electrolysis; and it has been asserted that the difference in the molecular arrangement of the *same* materials—due either to manufacturing methods which result in lack of homogeneity, or from the unequal application of force as stress that changes the arrangement of the fibres—is sufficient to produce voltaic destructive action.

Mill-scale.—In rolling iron or steel, the scale sometimes left upon the surface of the metal, and known as "mill-scale," has been analyzed as sesqui-oxide of iron, Fe^2O^3, the same chemical composition as ordinary iron-rust, and it seems further to possess to the same marked degree the capacity for absorption and condensing moisture and oxygen, producing corrosion and decay, and setting up galvanic action, the effect appearing in rust-cones pitting and eating the metal.

It is asserted that where mill-scale is left upon plates of

steel its effect upon the neighboring bared metal is as strong and continuous as copper would be in its galvanic action.

Overwhelming testimony and positive evidence have proven the following facts:

1st. That rust and mill-scale exert a most destructive action upon iron and steel.

2d. That where moisture and carbonic acid gas accumulate in considerable quantities, the rapid destruction of ferric bodies follows.

3d. That rusting, once started, progresses rapidly even under what seems a perfect protective covering.

4th. That if a covering can be found which will prevent the penetration of moisture, the perfect protection of the metal is assured so long as the covering remains intact.

In 1882 exhaustive experiments were conducted by authority of the British Admiralty, resulting in the following conclusions:

(1) That no pitting occured in mild steel when freed from mill-scale; (2) that the loss of weight from corrosion of clean mild steel and clean iron did not differ greatly; and (3) that the action of mill-scale is considerable and continuous, and equal to a similar quantity of copper in its corrosive action due to galvanism.

In long tunnels in which accumulations of carbonic acid gas and moisture are found, and as exampled by the Arlberg, St. Gothard and Musconetong tunnels, the life of iron or steel work is very brief, and a renewal every few years has been a necessity; in the last of these, it is reported that the 76-lb. steel rail was removed after five years' service and was found to have lost more weight by corrosion than by use.

The continuous action of rust is clearly shown by a report to the French Naval Office as to the effect of rust upon several torpedo-boats which had never been put into commission, but were laid up under cover and painted at intervals. An inspec-

tion showed that the plates under the paint were so corroded that the blow of a testing-hammer was sufficient to puncture them, and that large areas under the paint-film were so affected. This same effect of the continuous action of rust has been observed in the repair of numerous bridges and other structures, when the metal was found entirely destroyed under the paint-coating. A large truss-roof that was kept constantly painted having failed, it was found that the metal was simply rotten with rust under the paint, while no appearance of the instability of the structure from this cause was apparent to the eye. The same result is recorded by builders in the case of floor-beams which were practically eaten away below the paint-surface.

A recent investigation by Mr. D. H. Maury, of the electrolitic injury to the metal of the Peoria, Ill., stand-pipe is of great interest, and is given as follows:

"On March 30, 1894, the water company's steel stand-pipe on the West Bluff burst, killing one person and injuring 15 others, one of whom died later from his injuries. Upon examining the wreck of the stand-pipe, the writer at once noticed a peculiar pitting of the inside of the vertical sheets, and the appearance of these pits was so different from that caused by any ordinary oxidation that he was soon almost positive that they were due to electrolytic action. A similar stand-pipe on the East Bluff was drained, and was found to be similarly pitted. The whole inner surface of the vertical shell appeared to be thickly covered with blisters, resembling in outward appearance the tubercles sometimes found inside of old cast-iron mains.

"This blistered covering, which was almost as thin as paper, was composed entirely of oxide of iron, and on brushing it away with the finger-tips, the black paint with which the stand-pipe had been originally coated would be found beneath it.

"The black paint was oftentimes almost unbroken, or at least, very slightly cracked. When the paint was brushed off, the pit would be disclosed, considerably smaller in area than the surface covered by the blister. The surface of the metal in the pit was perfectly bright and clean, and its fibre was clearly discernible.

"Many of these pits were more than $\frac{1}{8}$ in. in depth. They were slightly more numerous in the West Bluff stand-pipe, and were in both generally larger and deeper on the lower courses of the vertical shell. . . . The East Bluff stand-pipe was distant about 60 ft. from the street-railway line on Bourland Street. The West Bluff stand-pipe was about 700 ft. distant from the railway line on Knoxville Avenue. Both standpipes were more than a mile from the power-station, and were negative to the rails. The electrical examination relative to the stand-pipes was conducted mainly at the East Bluff standpipe, which was still in service. A flow of a part of the current from the railway line was clearly traced through the earth to the anchor-bolts which held the stand-pipe to its foundations, up these bolts and into the steel of the shell, and through the shell and from its inner surface to the projecting section of the 16-in. flanged cast-iron pipe which served as both inlet and outlet, and which connected the stand-pipe to the water-mains. The current was then traced along this pipe and along the mains to the power-station. The deflection of the volt-meter needle was clearly traced to the railway current, being especially influenced by the one or two cars on the line beyond the stand-pipe on Knoxville Avenue, and when the cars stopped running at night, the movement of the needle ceased. Where the current left the inner surface of the shell to pass through the water of the inlet-pipe it made the pits already described. These stand-pipes and the inlet-pipes were negative to the rails, and are striking examples of electrolytic pitting under such conditions."

From the history of the Peoria stand-pipe, it having been noted that the specifications called both for iron and steel as structural materials and desiring to ascertain whether galvanic or battery action might not have been the result of the iron and steel in contact in the presence of moisture, the author wrote Mr. Maury, receiving a reply in which he stated that he did not think anything but steel plate had been used in the construction of the stand-pipe, except the rivets, and possibly the ladder and some connections; that careful investigations looking for battery action were made, but this action had not been substantiated.

Cleaning the Metal.—It having been shown and demonstrated that it is of prime necessity to prevent the commencement of the rusting process in its incipiency, and that the first consideration is to provide for the thorough cleaning of the metal before an attempt is made to give it a protective covering, it is in order to discuss the methods employed for this process of cleaning or preparation for painting.

For this purpose there are three processes in vogue and in general use. One is by "pickling"; another by the use of the sand-blast, and a third and more general method is by scraping and cleaning with wire brushes.

The pickling process consists in the submersion of the plate or shape in a bath of hydrochloric or sulphuric acid for a period of one-half to twenty-four hours, and afterwards neutralizing the acid by the use of lime, the lime then being cleaned off. The proportions of acid to water range from 10 to 19 parts of water to 1 of acid, the latter being the formula adopted by the British Admiralty. Such a method of cleaning plates, while reasonably economical and convenient, and fully effective when carefully performed, is open to the objection that any carelessness upon the part of the workmen is sure to produce results which are worse than the proposed cure. The second method of cleaning metallic surfaces is a mechani-

cal one, sharp-grained sand being employed under about 15 pounds compressed-air pressure at the nozzle, to cut away the rust and mill-scale, by being directed to the desired point from the end of a rubber tube or hose. While a certain method of cleaning when intelligent care is exercised, and the penalty for negligence not being so severe as where acid is used, the objection recorded to the use of sand is that a special building must be provided, from the fact that, unless the sand is confined, it is likely to prove damaging to machinery and become generally a nuisance.

The last and most popular method of cleaning plates and shapes is by the use of scrapers and brushes, either by hand or mechanically, electric revolving brushes being considerably used of late. The loosened material is wiped away with oiled waste or rags. Nearly all of the larger bridge-works clean their shapes in this way. The objection to this is that although the surfaces may seem bright and free from rust and scale, under a glass it will be seen that only the microscopic metallic points have been burnished, the depressions showing minute rust-specks which have not been touched by the scraper or brush, and may therefore become points or foci for corrosion. For these reasons, it would seem that specifications for the cleaning of metals should be drawn to include the use of the sand-blast, the cost of which is about the cost of a coat of good paint, and is said to be about $1.50 per ton of metal, exclusive of handling. During its evolution, the time at which the metallic member should be cleaned and primed is of great importance. In an investigation of this question, a testing-bureau, having a wide experience and facilities for observation, writes as follows: "In rolling a plate, a slab is drawn from the heating-furnace or soaking-pit, and it passes through the rolls. As it is being reduced, salt is thrown upon the slab; it causes a loud explosion, and loosens the scale formed and a steam-jet is turned on the slab, which blows

this scale off, so the finished plate comes with no scale upon it to the cooling-beds. In the rolling of angles and similar shapes it is not possible to do this. Therefore, there is more scale upon the angles than on plates. After rolling, shapes are as a rule stacked immediately upon loading-beds preparatory to shipment, it being against the mill's policy to hold material any longer than it is necessary to get cars and to load. Shapes after they come from the strengthening-press, which is directly after cooling, are not under cover. In case of plates, the conditions are different. After the plates are rolled they have to be laid off and sheared to size, and then stacked up awaiting shipment. In the majority of cases this is always under cover. Open cars are nearly always used in shipping steel, on account of the convenience in loading from cranes and also on account of the variation in lengths." The above explains the processes and evolution at the mills, and in order to arrive at the condition at which the material reaches the shops, inquiry was made of a large boiler and metal-working establishment, located from 600 to 700 miles from the point of metal-supply. They write: "We find very little rust, mill-scale, or grease on any of the sheets coming from the mills; though we must confess we find much more now than we used to heretofore. . . . There is a big difference in the steel plate from the different mills; there is a gloss or finish upon some, while from another mill they appear red, as though they were rusted. Now any of these plates will stand the weather without being injured or rusted, especially the ones best finished, and it is not necessary, in our opinion, to paint or oil the plates at the mill. The effect of rolling plates after they were painted would be to scale off much of the paint." From such testimony it appears that, under ordinary circumstances, it is not necessary to protect plates at the mill by painting or priming, and that at the shop the mechanical work of rolling to radius, as for boiler and stand-pipe plate,

and the punching and handling of untreated plates and shapes, as well possibly as the jar of railway transportation, and the several handlings, loosen more mill-scale than enough to compensate for any rusting in transit, and that therefore the proper time to clean and prime is at the shop, after the mechanical work has been completed, and immediately before shipment to the point of erection, any grease which may result from the machining being also subject to removal at the same time. The facilities for cleaning and painting being usually superior at the shop to those likely to obtain at the point of erection, is another consideration in favor of shop-cleaning and priming. Structural metal, when carefully cleaned of all rust, mill-scale, grease, and dirt, should be immediately protected by some covering as nearly impervious to moisture as possible in order to prevent further corrosion from chemical and galvanic action.

Zinc Coating.—It has been found that the application of molten zinc, called "spelter," as a bath, forms a coating which is electrically positive to iron or steel, and which in the presence of galvanic action results in the corrosion of the zinc and the protection of the ferric body. Such a coating is very effective, but with the larger plates, where the dipping is done by hand, the process is very expensive, $\frac{3}{8}$ in. plate being the thickest material so far galvanized for practical purposes, the cost being from $14.00 to $16.00 per ton. Besides the expense, unfortunately the process reduces the strength of plates and shapes to an extent that galvanized metal is generally considered as being "rotten" and unfit for use where certain and considerable strength is required.

Again, it has been asserted that water in galvanized receptacles or reservoirs becomes unfit for use, which, if true, would debar this method of protection either for the towers and members where strength was required, or for the tank, where the storage of water was the purpose of the structure.

A small municipal water-supply plant in use in California has two small galvanized tanks in service, which seem to have given satisfaction.

"**Oxidized Plates.**"—Another method of treating steel or iron plate for protection against corrosion, popularly called "oxidizing," has been accomplished in several ways with satisfactory results, the effect being produced by heating the metal, and afterwards subjecting it in a furnace to the action of mingled steam and carbonic acid gas, resulting in the production upon the metallic surface of a coating of the black oxide of iron, Fe_2O_3FeO.

It is claimed that the same result has been obtained by coating the metal with a mixture of red oxide of iron, containing an almost equal amount of silica and in a solvent of resin-oil, and afterwards heating the metal to a bright red. It is also claimed that the metal, heated to about 300 degrees Fahr., and immersed in an asphaltum mixture of the same temperature, will produce the same black oxide coating, but in this case it would seem that the plate must first have commenced to rust naturally, to produce the change from red to black oxide. In some of these processes, the change in the strength of the material is not more than that which would be produced by annealing, but in the first of these methods it is certain that the iron or steel is permanently expanded, which would be a certain advantage. The protective power of the black oxide film or coating is shown from the record of an iron column, said to have been erected at Delhi, India, about 900 B.C., and which is 60 ft. in height and weighs about 17 tons. After the lapse of ages, the surface is free from rust and otherwise unaffected by weathering.

Japanned Plates.—A permanent, hard and enamel-like coating, capable of successfully resisting the effects of corrosion, is known as "japan," and is produced by treating the article to be protected to a composition consisting of asphalt

and linseed-oil, as a base, with copal resin, thinned with turpentine, subjected afterwards to a slow heat in an oven or furnace, a process of baking. Trays, ornaments, door-locks and knobs, and small articles have been successfully treated to this process, and of late, experiments upon a larger scale have been made.

Practical Considerations.—While the adoption of such processes is known to afford more effective preventatives to metallic corrosion than any other method of covering so far developed, the effect upon the metal itself, the cost and inconvenience of operation, and the necessity of especial appliances would seem to debar such means from practical and general use for the protection of structural material, frequently in heavy masses; depending for its usefulness upon its certain and known strength, and whose manufacture, commencing at the mill, continuing at the shop, and possibly proceeding at remote points of erection, seems to permit the employment of no means which is not simple, convenient, speedy, and economical, which conditions are more nearly fulfilled by the protection afforded metals through paint-films, and it would therefore appear that, comparatively, they are the best protective coverings for iron or steel. As such agent, the records of the past leave much to be desired, and it should therefore be the serious effort of all engineers or other scientists, both chemists and physicists, to continue in an effort to develop this protective agency to the highest attainable degree.

Paint-films.—Paint is used for purposes of ornamentation as well as for protection, but only in the last of these functions will it be considered here, where the practical, rather than the æsthetic, is the prime consideration.

Paint is a film of one or more coats or thicknesses, which may be applied or spread with a brush over any surface, and

primarily consists of a liquid as the vehicle or medium, with which a base, or pigment, is in combination or solution.

A perfect paint should be tenacious; non-corrosive; elastic; impervious; of easy application; of reasonable covering and drying qualities, and of comparative economy.

The usual causes of the destruction of the paint-films when applied to such structures as metallic iron or steel tanks are expansion and contraction of the metal; sand or other sharp particles; or rain and sleet, contained in gusts of wind impinging upon the paint-film; the chemical and galvanic effect of light and heat, in the presence of moisture and gases, and acting upon the paint-substances; the lack of adhesion of the film to the metal, usually caused by the presence of moisture upon the metallic surface previous to the application of the film, resulting in "peeling," and finally the destructive action of the water enclosed in the tank upon the oil, causing swelling, shrivelling, disintegration, and a slumping away of the film.

Linseed-oil.—However much individuals may disagree as to the character of the pigment, linseed-oil as a medium or liquid vehicle, which has been used since the remote ages, continues the standard of efficiency.

Linseed-oil is a product obtained from grinding flaxseed to a coarse meal, which is heated and sacked, and being placed under powerful presses, the oil is extracted in a crude shape, and is refined by sedimentation and filtration extending over a period of from one to three months, becoming "raw" and "commercially pure" linseed-oil, costing from 55 cents to 75 cents per gallon.

"Boiled" linseed-oil costs a little more, and is produced by heating raw oil to 400 or 500 degrees F., at which temperature the vegetable matter of the oil is attacked, at which stage from 1 to 3% of either litharge or the red oxide of lead, sometimes with a small quantity of the oxide of manganese, is

added. Raw oil requires from five to six days in drying, while the boiled oil dries in about one-fifth the time.

No other known oil has the power for absorbing oxygen that is possessed by linseed-oil, but in the process it has been shown by Muelder that the oil gives off carbonic acid, acetic and formic acid, and possibly water-vapors, the slow escape of which probably accounts for the well-known porosity of the dried film, and on account of which the film has remarkable absorbent capacity, acting like a sponge in the presence of moisture, which Dr. Dudley considers the primary cause of the decomposition of the material, although not satisfied that the water itself is the cause of the decay.

Like other vegetable fixed oils, linseed-oil contains glycerine and liquid acid fats. According to many authorities, these fats in the presence of oxides, especially lead, produce salts by the combination of the acid fats with the lead of the oxide; saponify, resulting in metallic soaps. Amongst others, Prof. J. Spennrath combats this theory with many valid arguments, amongst which he asserts that "if we should treat any soap with diluted acid, which is capable of dissolving the metallic oxide contained therein, it is decomposed, and the fatty acid separated. The latter then swims in the liquid. A dried oil-paint can never be dissolved by diluted acid in this way." Again, "a weak alkalized liquid, for instance, a one per cent. soda solution, dissolves after a prolonged application any dried-up oil-paint coating. We then obtain the coloring matter what was used in an unchanged condition. A real soap cannot be decomposed by a soda solution."

Prof. Spennrath admits, however, that the rapid effects of oxidation produce more or less effect upon any oxidizable pigment, and several other recognized authorities assume, in the case of at least one such pigment—the red oxide of lead—that a chemical combination is produced, analogous to sapon-

ification, but with also a cement-like action, the substance "setting" into a compact mass during a short space of time.

Linseed-oil, then, alone or in combination with some inert pigment or substance, absorbs oxygen rapidly and in considerable quantities, wherever found, at the same time throwing off volatile gases, becoming porous and absorptive as it hardens into a tenacious, elastic vegetable gum; while in solution or combination with active mineral oxidizable compounds, a radical change takes place, the resulting substance being analogous to a metallic salt or soap, but evincing cement-like properties.

Pigments.—Of the *elementary* substances as a base of paint mixtures, it is generally conceded that Carbon C, as lampblack (or graphite), or the hydrocarbon asphaltum has given the best results for a metallic protective covering, while in the opinion of many the metallic oxides as red oxide of iron, (Fe_2O_3) and the red oxide of lead (Pb_3O_4) give equal or better results. These substances have been used singly, in combination with each other, or mixed with some of the "inert" pigments, such as silica, kaolin, talc, whiting, gypsum, etc. Comparisons, endeavoring to show why certain of the many pigments should *not* be used, have been so often made by eminent scientists that it will be the attempt of the author to give some reasons for the faith that is in him as to why certain of these bases *should* be used upon metallic structures, such as stand-pipes, not affected by heat or by sulphurous gases.

Before the American Society of Mechanical Engineers, June, 1895, Mr. M. P. Wood, a member of the society, read a paper entitled "Rustless Coatings for Iron and Steel," which is remarkably clear and interesting, and from which is quoted the following:

"**Red Oxide of Lead,** Pb_3O_4 (Minium).—This oxide is found native in various parts of the world, mixed with other

ores of lead, and probably resulting from their oxidation. In some localities it accompanies cerusite or white-lead ore.

"When prepared for analysis, or when the commercial article is freed from the protoxide by digestion with a solution of acetate of lead, it contains 90.63% of lead and 9.37% of oxygen, numbers agreeing exactly with the formula Pb_3O_4.

"It may be regarded either as a compound of the protoxide and peroxide of lead $PbO.PbO_2$, or perhaps of the protoxide and sesquioxide, $PbO.Pb_2O_3$, analogous to the magnetic oxide of iron. Its specific gravity ranges from 8.6 to 8.94.

"The commercial red oxide of lead is formed when the protoxide is kept at a low red heat for a considerable time in contact with air; also, after the previous formation of hydrated protoxide and basic carbonate of lead, when lead shavings are strewn upon the water, the vessel being loosely covered and set aside for some months, the formation of red lead taking place upon the surfaces of the lead exposed to the air. . . . Commercial red lead contains all of the foreign metallic oxides—such as the oxides of silver, copper, and iron—with which the *massicot* or *litharge* used in preparing it is contaminated. It is also adulterated with red oxides of iron, boles, or brick-dust; these substances remain undissolved when the red lead is digested in warm dilute nitric acid; boiling hydrochloric acid extracts the sesquioxide of iron from the residue. . . . The use of red lead as a pigment is possibly of earlier origin than any of the oxides of iron, ochres, and other substances, natural or artificial, of which we have any record, unless it be asphaltum or lampblack. The many miscellaneous pigments which have come forward, been tried, and found wanting in some one or other of the qualities which constitute a good paint are almost numberless. There is no other color-pigment whose use as a protective covering to wood, brick, stone, or metal has been so uniformly satisfactory and successful as red lead, and any failure to fulfil its mission

can be traced directly to some agency foreign to the lead itself, used either in its preparation or in the methods of its application."

A paper read by Prof. A. H. Sabin, before the Boston Society of Civil Engineers, November, 1899, says of the red oxide of lead: "There yet remains to be described one other important pigment, red lead. This is entitled to a place in a class by itself, because it is intermediate between the paints, which it resembles in being used mixed with oil, and the cements, which it resembles in its process of solidification. It is, in fact, a powerful basic substance, and combines chemically with the oil, forming an insoluble, hard, tenacious mass, in which the uncombined particles of the excess of red oxide are imprisoned. This is what constitutes the protective film when a red-lead paint is dry."

By some authorities it is claimed that in the chemical combination the glycerine, as well as the acid fats, is changed by the lead oxide, volatilization of the glycerine being prevented, but in oxidizing through the process common to all linseed-oils, the mass is rendered insoluble, elastic, and adhesive; but it seems very probable that the glycerine, not being a stable product, soluble in water and volatilized by heat, acts as described by Muelder, the film being rendered more or less porous by the escape of the gases.

Litharge mixed with commercial glycerine to a pasty mass takes a most hard and tenacious "set" when exposed to the action of the atmosphere for twenty to thirty minutes.

It is stated by Wood that, during the process of setting, red led and oil will oxidize the surface of clean iron or steel, forming the black oxide of iron which is non-corrosive. It is also believed to be a fact that where moisture exists upon the metallic surface, the oil and lead rapidly absorbs this in the chemical change requiring oxygen wherever found.

These estimable qualities, however, are offset to a certain

extent by the well-established facts that, on account of its specific gravity being far in excess of that of the oil, when mixed and spread upon perpendicular surfaces, the paint "runs" or "sags," the pigment separating from the oil, the coat producing a streaked appearance and not affording an even covering, and it would therefore seem that its use should be confined to metallic plates and shapes before assembling and where the coating can be applied while the member is horizontal or nearly so.

Again, owing to the rapidity of oxidation, the red lead and oil sets so quickly that it is of difficult application, but this objection can be partly overcome by an addition of a carbon-pigment, such as lampblack, which is an impalpable powder, practically indestructible, in a measure elastic, with the power of repelling moisture, and itself one of the best-known preservatives of metals, but comparatively useless when applied alone, from a fault in an *opposite* direction; that is, it takes *too long* to dry.

In conjunction, these two pigments modify the opposite objectionable properties of each, while the fine carbon-powder assists in filling any voids in the mass, due to imperfect combination.

In the manufacture of such paint it is a prime necessity that, to produce satisfactory results, each ingredient should be chemically pure, and the degree of purity will determine the relative efficiency. Suitable proportions have been found in 20 pounds of red lead, 1 pound of carbon as lampblack to 5 or 6 pounds of raw linseed-oil. The bulk will be about 1 gallon, with a covering capacity of about 50 square yards of surface for the first coat, the film being approximately .002 of an inch in thickness. The cost will be about $1.50, and the amount paid for labor in spreading will run about 5 cents per square yard where the services of an experienced painter are employed.

While the preponderence of evidence is in favor of the use of red lead in oil for protective coatings for iron and steel, numerous failures are recorded, but as a comparison of evidence might be continued *ad infinitum*, such a task will not be attempted here, further than to mention the results of a series of tests, extending over two years, and made by Prof. Sabin upon steel plates coated with a wide variety of paint covering, the samples being afterwards immersed continuously and subject for two years to the action of both salt and fresh waters. Prof. Sabin's conclusions, represented in a paper read before the Engineers' Club of Philadelphia, May, 1900, were that "the character of the pigment in a majority of cases made very little difference: that oil-paints did not withstand the action of the water as well as varnish-paints," but that "red lead stood better than any of the oil-paints. There is no question about it. It did not stand as well as many varnish paints. It did not stand as well as some varnishes without any pigment in them."

Structures are not as a rule subject to such action of the water as took place in Prof. Sabin's experiments, and while these were very carefully made and recorded, certain results where metal plates were submerged would not necessarily have a distinct bearing where a structure is subject only to atmospheric influence; but in view of the fact that such structures as tanks, intermittently or continuously filled with water, are the prime subject of consideration here, his experiments are of considerable value.

Asphaltic Varnish.—Varnish differs from paint only in the base—the medium, linseed-oil, remaining the same. In varnish, the pigment gives place to various resins, dissolved in the spirits of turpentine, a volatile oil. These resins are of vegetable origin, and are classed as "recent resins," the resinous gum of a recent period, and "fossil resins," the volatilized gums of trees long buried in the earth. Varnish resins are

largely found in Africa, South America, New Zealand, and the East Indies. The general process of varnish manufacture is the heating, in a suitable receptacle, of the resins to from 600 to 800 degrees F., at which point the resins melt, being decomposed by the heat.

At this point, hot linseed-oil is added, and the contents stirred until fully combined; after cooling, the mixture is dissolved or diluted with spirits of turpentine, to permit the proper flow of the varnish under the brush. The greater amount of oil used, the greater the elasticity, tenacity, and toughness, and the less brittleness, which are desirable qualities where the varnish coat is subject to mechanical injury. In addition to the vegetable resins, a "mineral resin," as it has been called, or asphaltum, is often used. Its oil, by dry distillation, is of a yellow color, and said to resemble closely the oil of amber. Used in considerable quantities in the manufacture of varnish, it exhibits remarkable non-drying qualities, but its compensating advantages are its cheapness, elasticity, tenacity, durability, and insolubility.

Prof. Sabin gives the following why varnish is better than oil: "The reason why varnish is better than oil is that it is more durable, smoother, and more brilliant, and because the resin dissolving in the oil makes it harder; it makes a film that is harder, and still retains a high degree of elasticity—not so much elasticity, perhaps, as the original alone, but a very high degree of elasticity; and it is very much more impervious to moisture than oil."

From a paper read June, 1895, by Prof. A. H. Sabin, before the American Society of Civil Engineers, the following is quoted:

"It has long been known to varnish-makers that the fossil resins known as copals, such as the New Zealand kauri, when added to asphalt-varnishes, improve their durability. This is probably partly owing to the fact that such compounds are

of greater density, as the resin dissolved in the oil and asphalt tends to make a more compact substance, and partly because it increases its electric insulating power, also in considerable measure because such a resin is very indifferent to the action of sulphur-gases. For all these reasons it seems to the writer that the maximum of durability is only to be reached by a compound of hard asphaltum, copal-gum, and linseed-oil, thinned, if necessary, with pure turpentine. It is of the highest importance that the oil employed should be so refined as to have its non-drying constituents removed, so as to avoid as much as possible the use of dryers. This is of more importance than in a pigment and oil-paint, because the most obvious thing about asphalt is mentioned in the observations of M. Riffault, made some thirty or forty years ago, that 'asphalt destroys the drying quality of oil.'"

This is due to the fact that, being a viscous substance, it closes the pores of the oil and thus obstructs the entrance of air and moisture, which is also the cause of the great durability of such compounds.

Not only is it necessary to have the most suitable materials in such proportions as experience has shown to be best, but the ingredients should be compounded in the most approved manner.

Long experience has shown that there are certain temperature-curves to be followed in combining certain materials, differing for different compounds, a departure from which injures the durability of the resultant compound. The upper parts of the curves approach dangerously near to the decomposing point of the oil, and it has been found that a suitably refined pure oil has that point more than 100 deg. F. higher than common oil; it is on this account, also, important to use the highest skill in the manufacture. The choice of ingredients is of less importance than their proper proportion, and this again is of no more value than the use of the best process

of combination. Against the use of varnishes upon metallic surfaces, it has long been pointed out that, on account of the volatile properties of the medium, either turpentine or benzine, its rapid evaporation causes a fall of temperature, causing a deposition of moisture upon the surface, which acts deleteriously upon the resin or gum of the varnish, while preventing the proper adhesion of the film to the metal, and possibly causing the commencement of the corrosive action of moisture upon the metallic surface.

The cost of a well-prepared asphaltic varnish, of pure materials, will be about $1.50 per gallon, which will cover about 40 sq. yds. of surface, one coat.

Application.—It is generally conceded that two coats of good paint will last at least three times as long as one coat, and that the first, or priming coat, is of especial importance.

In the prize essay of Prof. Spennrath, Director of the Technical School at Aix-la-Chapelle, upon "Protective Coverings for Iron," his conclusions are that: "It is therefore advisable, in putting on iron coatings, to prime with a paint as heavy as possible and have the upper coat rich in oil." The specific gravity of red lead being shown to be about 9.0, it is the heaviest known pigment in use in the preparation of paints.

In a number of exhaustive tests, Prof. Spennrath distinctly traces the bad experiences with red-lead coatings to the action of heat, under which conditions the metal expands, the paint-skin remaining hard and brittle, a severe stretching takes place, cracks and rents develop in the paint-coating, and as a consequence rust appears. Where the atmosphere contains hydric sulphide, the red lead is changed to the sulphide of lead, according to Prof. Spennrath, to which he attributes the sole specific weakness of red lead as a pigment.

To sum up, in favor of the use of red lead and oil is its well-known high specific gravity and its peculiar chemical

property of combination, resulting in the production of a coating or film of a particularly tenacious, hard, and insoluble character, when not subject to great heat or sulphurous gases, which is seldom to be considered in connection with such structures as towers and tanks. The red-lead paint, however, lacks elasticity, resulting in the formation of air-cracks, and its porosity from the escape of volatile gases during the process of hardening seems to be well established. Moreover, its high specific gravity has the disadvantage of causing the pigment to "sag" or run away from the oil when being applied, resulting in streaking or imperfect and uneven covering, while its quick-setting qualities render this paint unsatisfactory and difficult to handle. This last tendency may be in part or entirely removed by the addition to the mixture of carbon, usually in the form of lampblack, which further aids, as has been shown, in diminishing the porosity offered as an objection to the use of lead and oil, while if the paint is used, as before erection, upon materials and surfaces which may be placed horizontally or nearly so, the pigment has little or no opportunity to settle out of the oil or "sag."

For all the reasons submitted, it would appear that as a priming coat, or first coat, red lead, lampblack, and linseed-oil, when applied upon iron or steel surfaces of structural material before erection, affords the best known protection to metallic corrosion; it is also a well-established fact that red lead, usually as a red-lead paste, is used in water and steam-pipe fitting to produce a close and perfect joint, and that when applied upon the laps of steel plate intended to be used in water-tank construction, the same tendency toward producing a water-tight joint is observed, and the use of this material for such purposes minimizes the most objectionable practice of making it necessary to resort to a natural or rust-joint to secure the necessary degree of tightness between the metal plates.

It also seems equally sure that suitable finishing coats should be provided and applied over the priming coat, and that this last film should be of small specific gravity, elastic, impervious to moisture, hard, and tenacious; it should be indifferent to sulphurous gases and electrically insulating, all of which properties seem to be fulfilled to a greater degree by an asphaltic varnish than any known varnish or paint composition. On account of its ease of application and quick-drying powers, it is particularly suitable for application upon structures being erected in the open air and exposed to the weather, while the characteristic of a volatile composition to produce a deposition of moisture is of no consequence when that moisture is not formed upon the metal itself, but upon a cement-like coating, which, besides, has a power for decomposing moisture by the absorption of its oxygen.

Either paint or varnish coats should, when possible, be put on under the most favorable atmospheric conditions, the best season being during the autumn, when the temperature is apt to remain more uniform, and when fogs and rains are less likely to occur. A suitable interval of time should be observed in order that the first coat should be entirely and completely dry before the second coat is added. In order to have the painter or contractor observe this, and to make sure that more than one coat is put on, the several coats should differ slightly in color, so that such neglect would be readily determined and corrected.

In the purchase of materials, the preference should be given old and long-established houses, whose reputation for quality is well known, and it should not be expected that the purchase of paint materials at less than market prices will be conducive of anything but the practice of adulterating the products.

In the application of the paints, which should have been selected with considerable care, only experienced and reliable

mechanics should be employed; in the long run, besides their ability to spread a smooth and regular coat, their experience will save sufficient materia., or make the same material go enough further, to warrant the employment of the skilled mechanic, if the selection of the individual is put upon a basis of first cost, rather than of comparative excellence.

Repainting.—Intelligent and systematic care should be given a structure continuously after painting, remembering that "an ounce of prevention is worth a pound of cure." Repainting should not be too long delayed, and at the first evidence of this necessity, the old paint should be carefully removed before the fresh covering is applied. In doing this, a strong caustic solution should be used to partially decompose the old film, and steel scrapers and wire brushes then employed to detach the coat. Immediately afterward, the metallic surface should be carefully washed down with water and dried, any deep-seated rust-spots or paint which it has been impossible to remove otherwise being burned away by the application of the flame from a painter's torch.

It stands to reason that the more care exercised in cleaning down to the metal, the better the results from the new paint coating to be applied, and the greater logevity of the metal.

CHAPTER XI.

SHOP-PRACTICE AND ERECTION.

Laying Out Work.—As soon as the metal sheets or plates for tank or stand-pipe work are received at the shop, they should be immediately and carefully unloaded and stored awaiting the earliest moment when they may be "laid out." This process consists in marking off the plates for shearing, machining, punching, and rolling.

The object of shearing or machining is to put a bevel-edge upon the opposite face of the plate where two plates are to be in contact, and in order that the thin edge so formed may be properly and easily calked after riveting and that a water-tight joint may thus be secured.

For the reason that such work upon heavy plates has been shown to exert a force tending to change the molecular arrangement of the metal, this shearing of plates is usually not permitted upon plates that are thicker than $\frac{3}{8}$ of an inch, all plates above that thickness being planed to a bevel by a machine.

In laying out, the rivet-hole spacing is indicated by marking with a sharp-pointed cold-chisel, the widths from centre to centre, or the pitch, having first been calculated as has been described and explained.

Realizing that a greater comparative efficiency of joint-strength may be secured, with fewer rivets and wider spacing, where the largest possible rivet is used, this inclination is sometimes stretched to the limit, the requirement for tight-

ness of joints, as in stand-pipe work, being considered as having been provided for in the natural tendency of such joints to close by rusting after erection, and to what extent this practice is considered legitimate may be inferred from the following, taken from an article on painting, and from Prof. Pence's work, "Stand-pipe Accidents and Failures": "The methods of painting stand-pipes are subject to as much variation as in other exposed structural metal-work. Some require that the inaccessible surfaces shall receive two coats of red lead, while others allow the omission of paint from the faying surfaces of the seams to permit the joints to rust."

Again, according to recognized authorities, in forging a rivet, the color, indicating its temperature, should be about an orange red, and with steel rivets, with a tendency to rapid cooling, at this temperature the larger rivets, especially hand-driven, are so cold and tough before they are driven completely home and the head forged, that it is difficult to insure a perfect filling of the rivet-holes, and the requisite closeness of the joint, where rivets of large diameter are used, and for which reasons, in preparing the table given in the chapter on Riveting, these considerations were given weight. In the mention of this table, it may not be out of place here to refer to the dimensions and relative strength of the double-butt strap-joint, and to point out that while fully recognizing that the full strength of such a joint has not been developed, the necessity for such excess strength over and above all the other joints, both single, double, and treble riveted, did not seem necessary or particularly desirable.

Machining: Punching and Rolling.—After the plates are laid off and bevelled, the punching of rivet-holes should be done, and away from the surfaces to be in contact. Plates not exceeding $\frac{5}{8}$ inch in thickness may be punched with sharp and well-conditioned punch and dies, either singly or preferably by a power-machine employing several such

punches or dies, properly spaced. The area of the rivet-hole should be about $^1/_{16}$ inch greater than that of the rivet proposed to be used.

Plates having a thickness between $\frac{5}{8}$ and $\frac{7}{8}$ inches should be punched $^1/_{16}$ inch less, and reamed out; while plates over that thickness should be drilled from the solid sheet.

While it has been shown that for tank work, plates, regardless of thickness, can be connected in a more mechanical fashion by requiring the horizontal seams to be a lap and the vertical joints a strap connection, for reasons of economy, the lap-joint is used and will probably continue tin use for connecting all plates, for both horizontal and vertical seams, where the thickness of the plates are less than $\frac{1}{2}$ inch, and possibly a thickness of $^{13}/_{16}$ inch should be considered as the maximum permissible thickness for the use of a lap-joint. In order to make the lap-connection, a corner of the plate has to be heated and drawn out to make the joint where three plates come together. This drawing out after heating is called "scarfing," and is objectionable, both on account of the unmechanical joint produced and as well as from the fact that this reheating and working of the steel reduces its strength, as has been explained in the chapter on the Physical and Chemical Properties of Steel.

When, from reasons of economy or other necessity, this reheating is permitted, that it may be as little objectionable as possible, it is recommended by authorities that the temperature of the metal, and which permits working, shall range between a heat which will ignite hard wood and the boiling temperature of water. In flanging or other bending, it is sometimes necessary to work over the metal in this way, but for bending sheets and angles to radius for tank work, heating is not necessary and should not be allowed, it being entirely possible to bend the metal to the required shape

when cold by passing it through powerful steel rolls; this is called "cold-rolling," and should always be specified.

Such rolling should invariably follow the work of bevelling and punching, better results being obtainable through such process.

Shop-assembly.—Immediately after rolling, the various separate parts of the structure should be assorted and "assembled," to insure a fair and satisfactory arrangement at the point of erection. Where the rivet-holes do not match perfectly in the assembled parts, the rivet-holes should be made to coincide and any eccentricity should be corrected by reaming out the hole and providing for a larger rivet.

After testing the several members during this "shop-assembly," each piece should be regularly and carefully marked, that no confusion may result at the time of "field" or final assembly.

Cleaning and Priming.—Immediately after testing and correcting the shop-work, the parts should be carefully cleaned of all dirt, grease, mill-scale, or rust, as has been explained, preferably by the use of the sand-blast, after which, as has been suggested, a coating or priming should be made with red lead, lampblack, and linseed-oil, and as soon as sufficiently dry for handling, the material should be carefully loaded into the cars, and consigned to the point of erection.

This class of work as above described is usually done by any well-equipped boiler-works, and the shop-cost is about $20.00 per ton, exclusive of painting.

During the progress of the work, independent shop-inspection should be insisted upon and carried out by an experienced and reliable inspector whose fee would amount to approximately 40 to 50 cents per ton of material, or about $1.00 per ton for complete inspection and test at both mill and shop.

Angles and other shapes, intended to form such a superstructure as a tower, are usually sheared, milled, and connected by riveting at a well-equipped bridge-works. The same precautions as to riveting and cleaning should be taken as with the tank work, and surfaces in contact and thereafter inaccessible should be given at least two coats of red lead and oil. Only connections should be made in the field, all other parts being riveted in the shop before shipment.

Preparation of Foundations.—To avoid what is known as "green masonry," as far in advance as possible before "field-work," the foundation masonry should be laid. The site of the structure having been determined, careful tests should be made to determine the character of the soil and to ascertain its bearing value. Such tests may be made by driving test-pits with such an implement as a post-hole digger, or by borings made with an auger of not less than 2-inch diameter. The auger-bit is welded into a short section of pipe; another short section is fitted with a cross-piece or handle, and additional sections, having suitable couplings, are to be prepared in sufficient number to permit the borings to be carried to a safe and satisfactory depth. As soon as expedient after such borings, and the design of a foundation to support the structure, excavations are made and the subfoundation or bearing prepared. The character of the connections for the anchorage having been designed, flat planks or boards should be connected in such a way as to form a suitable templet, which should be carefully laid off and holes of proper size bored. The anchor-rods having been provided, these are usually enclosed in old boiler or other tubes, slightly larger than the anchor-rods, and of approximately the same length or a little shorter.

The rods and tubes are inserted into the holes of the templet, which is then raised to the correct height or level and made fast with wooden props or stays. Each of the

washers of the rods are then carefully levelled and the rods plumbed, generally with a line and bob, after which the masonry is commenced and continued to completion, the tubes remaining in place until that time, when they are withdrawn, leaving a space about the anchor-rod, which allows slight adjustment of the rod to suit the connection when placed.

Upon the completion of the masonry, the templet and braces are removed, the rods tested and adjusted, and the spaces about them filled with cement grout, as thick as can be poured.

All series of levels taken should be carefully recorded, and should refer to a permanent "bench-mark" or datum. In this way, any irregularity during construction may be corrected and any subsequent settlement may be noted.

In the foundations for the usual tower, the templet for the rods and tubes is generally formed of a single plank, thick enough to prevent sagging, and which is accurately placed across the foundation-pit, buried flush with the earth, and frequently fastened or staked down to prevent disturbance. The rods are passed through suitable holes bored in this plank, levelled and plumbed. It is hardly necessary to remark that each of these foundation-pits require a separate plank.

Preliminaries to Erection of Stand-pipes.—The foundations being ready to receive the superstructure, provision should be made for carefully unloading the material upon its arrival, for which purpose, ordinarily, a short "gin-pole," with a metal hook or rope sling at its top, and guyed in a vertical position and adjacent to the transfer track is found convenient.

Great care should be taken to prevent bending any of the sections or rubbing or scratching the surface which should have been primed.

Arrived at the foundations, the sheets should be systematically placed, the bottom pieces and angles being nearest; the top pieces, cresting, etc., furthest away from the foundations.

Upon the top or face of the foundation, it is customary to place the kegs of rivets, which being of the same height, make a sort of platform upon which the bottom plates may be put together.

After these have been riveted to each other and to the circumscribing angles which fasten the bottom and shell, the tightness of the bottom is tested by pouring water upon the plates. If the joints are not found to be tight, they are further calked, or if the leak is due to imperfect or loose riveting, such rivets are cut away with chisel and sledge; the hole is reamed larger and a larger rivet inserted and driven.

Field-assembly.—These preliminaries having been observed, about the outer circumference of the foundations a slight, low dam of clay puddle or even of sand is constructed; into the area so formed is then slushed or poured a rich cement grout, sufficient to cover the face of the foundations and deep enough to entirely cover and hide the heads of the rivets upon the under side of the bottom plates. Having been quickly "floated" or levelled over, the bottom of the tank is lowered as rapidly as possible, by means of jacks or levers.

The separate sheets of the first ring are then set in position, being temporarily bolted to place and afterwards riveted.

As each sheet is placed, the surfaces in contact, or the joint surfaces, should be given another coat of thick red lead and oil, as should also the joint after riveting, that the rivet-heads may be entirely covered to prevent the formation of rust during construction and before the finishing coats of paint are supplied.

With the second and succeeding rings, a short "gin-pole" is first bolted to the top rivet-holes of the section below, and sheets are hoisted in succession and temporarily fastened with bolts until the entire circle has been so placed, when riveting is begun, the heating-forge being conveniently located in a travelling-carriage or "cage," moving along the circumference upon small rollers or trolleys as required, while the riveter, forming the field-heads with a forming-hammer, upon the head of which two men strike with sledges, remains upon the inside of the structure, all the workmen standing upon scaffolding, which is raised as the work proceeds, and which may consist of $2'' \times 2''$ uprights.

Upon the completion of the metal-work of the shell, the ornamental cresting or cover, the ladders and other fittings and trimmings are put in position; the tank then being ready for testing, is filled with water. Leaks along the seams are caulked carefully, but no caulking should be permitted upon leaks about rivet-heads, due to imperfectly-filled rivet-holes or loose rivets. Such rivets should be cut out with chisel and sledge; the hole reamed out and larger rivets driven. Such leaks are carefully marked while the water is in the tank and the repairs made after the vessel is emptied. No caulking or chipping should be allowed while the water remains in the tank. The hoisting of plates is usually done by hand, using a winch, from which a line passes through a block hung from a loop or hook on the "gin-pole," and to which is attached some form of tongs or "grab," which may be hooked into the rivet-holes of the sheet to be hoisted. A "riveting crew," or gang, consists usually of a foreman, who also personally does the caulking of seams; a riveter, generally an experienced boiler-maker; a skilful "heater," who heats the rivets to a forging heat and passes them in tongs into the rivet-holes, and three laborers, one of whom directs a heavy suspended weight against the rivet being

driven, while the other two strike in turn upon the hammer held by the riveter in forming the field-head. Two extra laborers are generally employed to work at the winch and to sort out material as directed by the foreman.

Such a crew will drive from 400 to 500 rivets per day of ten hours, at a cost of 3 cents each, or the entire cost of erection, including riveting, will amount to about $20.00 per ton of material. The scaffolding is left in place upon the inside of the tank until after testing by filling. The tank being tight, it is then removed. Instead of the scaffold as described, a floating scaffold is sometimes employed, which consists of a buoyant platform or float that is raised to position as required by pumping water into the tank.

Inspection.—After inspection and approval of the metal-work and the emptying of the water used in testing, the interior surfaces should be wiped dry with oily cloths, and the final coating or painting given, the scaffolding being removed as the painting proceeds from the top downward. In view of the fact that a heavy gale is liable to seriously affect the joints of the stand-pipe if empty, by straining the structure, immediately upon the drying of the paint, the reservoir should be filled with water and kept so filled until put into actual use as part of the water system.

Erection of Towers and Tanks.—In the erection of a tower, the pedestal-plates should be bedded in cement mortar about an inch thick. The first step toward erection is to conveniently place the columns and members of the first panel or section, and in such position that, with the aid of a stout gin-pole, blocks, tackle, and winch, the columns may be simultaneously raised to their vertical position and the horizontal members placed and temporarily fastened with bolts, to be subsequently riveted before proceeding with the next panel or deck.

As has been remarked, the field-riveting should be con-

fined entirely to panel-points or points of connection, all other rivet-work having previously been done at the shop.

The first panel having been secured, a smaller gin-pole is bolted to each of the columns or legs in succession, and the next vertical member is raised to its place and fastened by bolts until all of the column-sections are so located, when the horizontal and diagonal members are hoisted into position and secured. When the last or upper panel is in place, where the structure is surmounted with a platform, this is erected, from which work conveniently proceeds upon the girders, bottom, and subsequent tank-sections or rings, as has been described.

An approximate cost of such work is $25.00 per ton of material, varying with the local conditions at the point of erection.

Field-riveting and Machine-driven Rivets.—As the field-work consists largely of riveting the members together, the following, taken from the *Locomotive*, a paper published by the Hartford Steam-boiler Inspection and Insurance Company, may be of interest: "The driving of rivets is such a comparatively simple operation, that it might be supposed that it would be almost always well done. This is far from being the fact, however, and bad riveting is one of the commonest defects reported by our inspectors.

"The rivets may be too short, or too long, or too small; they may have heads that are too flat, or they may have projecting 'fins,' or they may not fill the holes, or the holes may not come 'fair' with one another. There are many ways in which riveting may be bad. . . ." In reporting a particular case of imperfect rivet-work in the same article, is the following: "The inspector found the rivets 'driven very low' —that is, the heads were entirely too flat. He had a number of these rivets taken out, and found that the holes in the two sheets did not come opposite one another fairly. This

defect is a common one, and it is very serious, both because it reduces the shearing-area of the rivet, and because it greatly increases the difficulty of making the rivets fill the holes perfectly. A shop that turns out work of this kind is particularly censurable, not only because the work itself is poor and weak, but also because the defect is not easy to discover, after the rivets are in place, and the owner of the boiler is therefore likely to be deceived by a fair external appearance, and to carry more pressure than the boiler can safely withstand. The inspector also found that the heads were not driven evenly over the holes, the centres of the heads often lying well towards the side of the rivet. This defect, although not so dangerous as the unfairness of the holes, would not be tolerated in a good shop having any pretense of turning out first-class work. It is very easily detected, even by one who has had little experience in inspecting; and there is no excuse for it whatever. . . . The only thing that could be done in the way of improvement would be to cut out all the rivets, ream out the holes until they should be true, and rivet them up again with larger rivets."

There are many reasons for the belief that a machine-driven rivet makes a much more satisfactory job than where a rivet is driven by hand, for the metal cooling rapidly, the greatest power and certainty is required to forge the head before the rivet material is too cold to work. Various types of power riveting-machines are now built whose motor force is either air, steam, water or electricity, affording a constant pressure throughout the stroke of about 80 pounds.

From comparative tests with both power- and hand-driven rivets, in Kent's "Mechanical Engineer's Handbook," is recorded the slip of plates pulled apart. In this it is shown that machine-driven rivets of equal diameter held twice as much as hand-driven rivets.

At the Gas Exhibition, held in New York about 1897,

samples of heavy plates riveted by both hand- and machine-work were split with a saw, and the rivets and holes shown in cross-section. All machine-driven rivets completely filled the rivet-holes, while the hand-work was seen to be very irregular. In his work entitled "Iron Highway Bridges," and in connection with suggestions for riveting, the following is given by Mr. Alfred P. Boller, M. Am. Soc. C. E.: "Power-riveting is so superior in all respects to hand-riveting that a higher unit of strain, by probably 10 per cent., can be used under the former system; so that if it is considered proper to strain hand-rivet work up to 13,500 lbs. per square inch, work riveted up by steam or hydraulic power can be safely proportioned on a basis of 15,000 lbs. per square inch."

So clearly is the superiority of power-riveting, that it is specified almost exclusively for boiler-work, bridge-work, and in fact for almost all shop-work, but its use in the field is comparatively limited and of recent date. In this connection, the *Engineering News* for May, 1895, publishes a description of a stand-pipe erected at St. Barnard, by L. Schreiber & Sons Co., of Cincinnati, Ohio, who used for the field-work a pneumatic riveting-machine, suspended from a hoist by the arm of a crane with mast in the centre of the shell. In response to an inquiry as to this work and as to the cost and efficiency of power field-riveting in general, Messrs. Schreiber & Sons Co. reply "that we have found pneumatic riveting much better than hand-work, especially so if the machinery is of the proper kind. We do this work under very high pressure and hardly believe (owing to the fact that the machinery required for this work is very heavy) that there is a great saving over hand-riveting. However, there is a little in favor of the machine-riveting."

The Logan Iron Works, contractors for a stand-pipe at College Point, L. I., used a pneumatic riveting-machine in driving some 75,000 rivets. According to information re-

ceived from the manufacturer of this machine, "not a single rivet had to be cut out or caulked, a most exceptional record which has not been equalled by any other machine. They drove 800 to 1200 rivets per day, depending on size. They tell me the cost of driving by machine was less than half that of driving by hand. Allowing three men and a boy on machine, at $9.00 per day and $4.50 for cost of running air-compressor and fuel, or $13.50 per day for crew, this makes a cost of about one to one and a half cents per rivet."

A quotation from a communication to the *Engineering News* from Mr. Freeman C. Coffin, M. Am. Soc. C. E., will be used in concluding this subject, and is as follows: "The rivets should be driven by steam or hydraulic power. This may seem radical, but I do not think so. I see no real reason why it could not be done with the suitable appliances. If field-riveting can be done by power in any structure, a stand-pipe is the best form, as there are continuous rows of rivets of about the same diameter, and the only especial form of appliance would be the yoke of the riveter, which would need to straddle a 5-foot plate. I do not believe that this is impracticable. I think it must hurt the feelings of any engineer to see two men with heavy sledges pounding away at a cool rivet, endeavoring to form a head on it. The usual result is a very thin, flat head, as the rivets are used as short as possible in order not to cause too much trouble if they happen to get cold before they are finished."

FINIS.

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